PROCEEDINGS

THE INSTITUTION OF CIVIL ENGINEERS

PART I
JANUARY 1955

ORDINARY MEETING

2 November, 1954

WILFRID PHILIP SHEPHERD-BARRON, M.C., T.D., LL.D., the retiring President, in the Chair.

The Council reported that they had recently transferred to the class of

Members

AKERS, RAYMOND LANGFORD, B.Sc. (*Birmingham*).
ASPEY, THOMAS HULME, M.Sc. (*Manchester*).
BATE, EDWYN ERNEST HOPE, C.B.E., M.C., B.Sc.(Eng.) (*London*).
BOOTH, ARNOLD GEORGE, M.Eng. (*Sheffield*).
BORLASE, ARTHUR, T.D.
CARTER, ERIC BRISCALL.
CLEMENTS, ERNEST WILLIAM FREDERICK.
HAJNAL-KÓNYI, KALMAN.
HAMILTON, ROBERT JOHN, B.Sc.(Eng.) (*London*).
HEANEY, ROBERT FRANK.
HOWE, EDGAR, B.Sc.(Eng.) (*London*).
HUGHES, GEORGE, B.Sc.(Eng.) (*London*).
HUNTER, LESLIE ERNEST, M.Sc.(Eng.) (*London*).
IRVING, THOMAS GAVIN, B.Sc. (*Glasgow*).
JOHNSON, WILLIAM JOHN BLOIS, B.Sc. (Eng.) (*London*).
KEER, EDWARD WINGROVE.
LAKEMAN, HECTOR GEORGE, B.Sc.(Eng.) (*London*).
LEGGE, HUGH GOUNTER HENEAGE, B.Sc. (*Witwatersrand*).
LEVIN, ABRAHAM ADOLF, B.Sc. (*Witwatersrand*).
LITTLE, GILBERT, B.Sc. (*Glasgow*).
LUMBARD, DONALD, M.B.E., B.Sc. (*Bristol*).
McGAREY, DONALD GRAHAM, B.Sc.(Eng.) (*London*).
McINTOSH, FRANK JOHN, B.Sc. (*South Africa*).
McMILLAN, THOMAS.
MADANPOTRA, GURMAKH SINGH, B.Sc. (Eng.) (*London*).
MAYCOCK, MARTIN GEORGE, B.Sc.(Eng.) (*London*).
MILNE, MAURICE, B.Sc. (*Aberdeen*).
MORRICE, HUMPHREY ALAN WALTER, B.A. (*Cantab.*).
MYERS, SYDNEY MYER, B.Sc.(Eng.) (*London*).
NORRIS, PETER JOHN.
POOLEY, HENRY POPHAM, B.A. (*Cantab.*).
PUGH, IFOR WYN.
ROBINS, ERIC GEORGE.
ROSE, ERIC OLDING, B.Sc.(Eng.) (*London*).
ROWNTREE, JOHN BURGESS, B.E. (*New Zealand*).
ROYDS, HAROLD GEORGE.
SUMMERS, MAURICE WILLIAM.
TAIT, ROBERT JAMES.
TAYLOR, REGINALD WILLIAM, C.M.G., B.Sc.(Eng.) (*London*).
WARD, JOHN WALTER, M.A. (*Cantab.*).
WATSON, JAMES KERR, O.B.E.
YOUNG, DONALD FRASER.

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and had admitted as

Graduates

AIREY, DONALD CHADWICK, B.E. (*New Zealand*).
ALEXANDER, COLIN ALSTON, B.Sc. (*Cape Town*).
ALLEN, ALEXANDER GORDON, B.Sc. (*Belfast*).
AMOTT, FRANK ALBERT COWPER, B.Sc. (Eng.) (*London*), Stud.I.C.E.
ANDERSON, DAVID GRAHAM, Stud.I.C.E.
ASHTON, JOHN ANTHONY, B.Sc. (*Leeds*), Stud.I.C.E.
ATTWOOD, JOHN HARRY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
BAIRD, JOHN, Stud.I.C.E.
BALCOMB, GEOFFREY WILLIAM, B.Sc. (*Wales*).
BARTLETT, JOHN VERNON, B.A. (*Cantab.*), Stud.I.C.E.
BATTERSEY, DOUGLAS, Stud.I.C.E.
BELL, JOHN STEPHEN, B.Sc. (*Leeds*).
BETLEY, GEORGE EDWIN, Stud.I.C.E.
BLACK, THOMAS JOHN, B.Sc. (*Glasgow*), Stud.I.C.E.
BLACKHALL, LAWRENCE BARNABAS BRYE, B.Sc.(Eng.) (*London*).
BODEN, PETER, B.Sc. (*Wales*), Stud.I.C.E.
BRADLEY, DONALD, B.Sc. (*Birmingham*).
BRADSHAW, IAN ERNEST, B.Sc.Tech. (*Manchester*).
BRADSHAW, MATTHEW, B.Sc. (*Glasgow*), Stud.I.C.E.
BRAZIER, JOHN GORDON, Stud.I.C.E.
BREIK, SAID, B.Sc. (*Durham*), Stud.I.C.E.
BREMNER, RAYMOND MUIR, Stud.I.C.E.
BRENTNALL, PETER HENRY, Stud.I.C.E.
BRIGHT, FRANK, Stud.I.C.E.
BRITT, GEOFFREY BRIAN, M.A. (*Cantab.*), Stud.I.C.E.
BROADHEAD, JOHN ANTHONY, Stud.I.C.E.
BRODIE, JAMES WALLACE, Stud.I.C.E.
BROOKS, GEORGE EWAN, B.Sc.(Eng.) (*London*), Stud.I.C.E.
BROWN, EDWARD ALAN HARVEY, Stud. I.C.E.
BROWN, MICHAEL ERNEST, Stud.I.C.E.
BROWNE, JOHN ROBERT GRIDLEY, B.Sc. (Eng.) (*London*).
BRUNTON, JOHN DAVID, B.Sc. (*Leeds*), Stud.I.C.E.
BURCH, JOHN PERCY COPPER, B.Sc. (Eng.) (*London*), Stud.I.C.E.
BURKE, THOMAS JOHN, Stud.I.C.E.
BUTLER, FREDERICK GEORGE, B.Sc. (Eng.) (*London*), Stud.I.C.E.
CAIRNS, DESMOND, B.Sc. (*Belfast*), Stud. I.C.E.
CALLAGHAN, SAMUEL JAMES, B.Sc. (*Belfast*), Stud.I.C.E.
CAMERON, IAN GEORGE DEWAR, B.E. (*Queensland*).
CANESSA, ERIC ALBERT JOSEPH, Stud. I.C.E.
CASH, GRAHAM HENRY, Stud.I.C.E.
CHADWICK, JOHN MICHAEL, B.Eng. (*Sheffield*), Stud.I.C.E.
CHAPMAN, DONALD JOHN, Stud.I.C.E.
CHAPMAN, MALCOLM HERBERT, B.Sc. (*Birmingham*), Stud.I.C.E.
CHERRY, ROBERT DENIS, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
CHU-CHEONG, CLIVE KENDALL, B.Sc. (*Wales*).
COLE, ERIC MAXWELL, Stud.I.C.E.
CONCHIE, WILLIAM GEORGE, Stud.I.C.E.
CLARKE, PETER JARDINE, B.Sc. (*Wales*).
CLEGG, JOHN HODGKINSON, B.Sc. (*Manchester*), Stud.I.C.E.
COMLEY, PETER, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
COOK, GEOFFREY RAMSAY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
COOLEY, ERIC HUMPHREY, M.A. (*Cantab.*), Stud.I.C.E.
COOPER, WILLIAM HOWARD, B.A. (*Cantab*).
CORRIE, JAMES GRAHAM, B.A., B.A.I. (*Dublin*).
CORSIE, OLAF JOHN, B.Sc. (*St Andrews*), Stud.I.C.E.
COWAN, JOHN CHAPMAN, Stud.I.C.E.
CRESSWELL, ROY CHARLES, Stud.I.C.E.
CROCKER, PETER, Stud.I.C.E.
CULLEN WALLACE, ANDREW ALASDAIR, B.Sc. (*Edinburgh*).
CURRIE, NEIL, B.Sc.(Eng.) (*London*).
CURRY, ROBERT GEORGE, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
DAVISON, ALAN ANDREW ELLIOT FORD, B.Sc. (*Belfast*).
DEAN, JAMES DOUGLAS, Stud.I.C.E.
DE BLAQUIÈRE, THOMAS EDWARD, B.A., B.A.I. (*Dublin*).
DENNIS, THOMAS EDWARD, B.Sc.(Eng.) (*London*), Stud.I.C.E.
DOEL. ALAN THOMAS, B.Sc.(Eng.) (*London*), Stud.I.C.E.
DONNELLY, MALCOLM CARL, B.Eng. (*Sheffield*), Stud.I.C.E.
DOUTHWAITE, GEORGE, B.Sc.(Eng.) (*London*).
DOWELL, JOHN RUDKIN, B.A. (*Cantab*).
DOWNEY, VICTOR MICHAEL, Stud.I.C.E.
DUDENEY, DONALD FREDERICK, B.Sc. (Eng.) (*London*), Stud.I.C.E.
DUGGAN, PATRICK JOSEPH, B.E. (*National*).

Durose, Bryan William, B.Sc. (*Durham*), Stud.I.C.E.
Ellis, Ivor Wynne, Stud.I.C.E.
Ericsson, Oscar Roy, B.Sc. (*Birmingham*), Stud.I.C.E.
Evans, Alan John Richard, B.A. (*Cantab.*).
Fairhurst, Leonard, B.Eng. (*Liverpool*), Stud.I.C.E.
Fargher, Sydney Edwin, M.A. (*Cantab.*), Stud.I.C.E.
Fisher, Roland Desvignes, Stud.I.C.E.
Ford, Laurence Mark, B.Eng. (*Liverpool*).
Forsyth, Donald, B.A., B.A.I. (*Dublin*).
Foster, Brian Wentworth, B.Sc. (*Manchester*).
Freer, Robert, B.C.E. (*Melbourne*).
Gambrill, Lewis Charles Ernest, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Geraghty, Adrian, Stud.I.C.E.
Geraghty, William Sidney, B.E. (*National*).
Gibbs, Leonard Thomas, Stud.I.C.E.
Gillies, George Albert Henry, Stud. I.C.E.
Goldsbrough, Derek Howard, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Goodwin, John Albert, Stud.I.C.E.
Grange, David John Hudson, Stud. I.C.E.
Green, Desmond William Burbidge, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Green, Rowland, B.Sc. (*Durham*), Stud.I.C.E.
Greer, Charles Ian Robb, B.Sc. (*Belfast*), Stud.I.C.E.
Grice, Alan Norman, Stud.I.C.E.
Griffin, William John, Stud.I.C.E.
Grigsby, Reuben Edward.
Grummitt, Claude Norman, Stud.I.C.E.
Gunawardana, Leonard Cyril, Stud. I.C.E.
Gupta, Surindar Paul, Stud.I.C.E.
Hadden, Ronald Newenham, B.Sc. (Eng.) (*London*).
Halstead, Roy Joseph, Stud.I.C.E.
Hammond, Burrell Victor, B.A. (*Cantab.*).
Harris, Martin Anthony, B.Sc. (*Durham*), Stud.I.C.E.
Hartwell, Raymond Leonard George, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Harvey, Percy, Stud.I.C.E.
Hasson, Raymond Rachamim Hasdai, B.Sc. (*Cape Town*), Stud.I.C.E.
Hay, David, B.Sc. (*St Andrews*).
Hayward, Roger Kendrick, M.A. (*Cantab.*).
Hayward, William Henry, Stud.I.C.E.
Heathcote, Neville Barrie, B.A. (*Cantab.*), Stud.I.C.E.
Heatley, John Samuel, B.Sc. (*Durham*).
Hebditch, John Anstey, B.A. (*Cantab.*), Stud.I.C.E.
Henson, Joseph Ronald, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Hill, Arthur Bradley, Stud.I.C.E.
Hipwell, Peter Daniel, B.Sc. (*Birmingham*).
Hislop, James Wilson, Stud.I.C.E.
Hogg, Denis Broadbery, B.Sc. (*Leeds*).
Holder, Brian James, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Holmes, Lawrence James Oliver, B.Eng. (*Sheffield*), Stud.I.C.E.
Hopkins, William Morris, B.A. (*Cantab.*).
Hughes, John, B.Sc. (*St Andrews*), Stud.I.C.E.
Innes, Kenneth William, B.Sc. (*Bristol*).
Jackson, James Keith, B.Sc.(Eng.) (*London*).
Jacobs, Alexander, B.Sc.(Eng.) (*London*).
Jainudeen, Tuan Tharik Rumi, Stud. I.C.E.
Janes, Geoffrey Morris, B.Sc. (*Cape Town*).
Jenkins, William McLaren, B.Sc. (*Glasgow*), Stud.I.C.E.
Jones, Frank Henry, Stud.I.C.E.
Johnson, Anthony Ernest, Stud.I.C.E.
Johnston, David Robert, B.Sc. (*Glasgow*), Stud.I.C.E.
Johnston, Frederick Gerald, B.Sc. (*Belfast*), Stud.I.C.E.
Jones, Dennis Carlton, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Jones, Evan John, B.Sc. (*Wales*).
Jouzy, Neddy Costandy, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Keast, Wallace James, B.Sc. (*Bristol*).
Kelly, William John, Stud.I.C.E.
Kemp, Charles Ian Taggart, Stud. I.C.E.
Kemp, John Brian, Stud.I.C.E.
Kerr, Geoffrey Brian, Stud.I.C.E.
Kerr, William David Ross.
Khanna, Prem Prakash, B.Sc. (*Glasgow*), Stud.I.C.E.
Kincaid, William, B.Sc. (*Glasgow*), Stud.I.C.E.
Kingaby, Norman Ford, B.Sc.(Eng.) (*London*).
Kinneen, Richard John Noel, B.E. (*National*).
Kirk, Donald Geoffrey, B.Sc. (*Leeds*), Stud.I.C.E.
Kulatunge, Don Gerard Percy, B.Sc. (Eng.) (*London*).
Kynaston, Robert Francis, B.E. (*Queensland*), B.A. (*Oxon*), Stud.I.C.E.

Lamb, Alfred Simpson, B.Sc. (*Glasgow*).
Lambert, Leonard Charles, B.Sc. (*London*), Stud.I.C.E.
Lawson-Williams, Ian Malcolm, B.Sc. (*Eng.*) (*London*), Stud.I.C.E.
Leach, George Austin, B.Sc.(Eng.) (*London*).
Leeming, Michael Brettargh, Stud. I.C.E.
Leeming, Walter Francis.
Lempriere, Norman Everard, B.Eng. (*Liverpool*), Stud.I.C.E.
Lennox, Robert Alexander, B.Sc. (*Glasgow*).
Leonard, Thomas John Gabriel, B.A., B.A.I. (*Dublin*).
Levy, Barry, B.A. (*Cantab.*), Stud.I.C.E.
Lewis, Albert Keith, B.E. (*New Zealand*).
Lewis, Conrad Walford, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Loose, Fred Walter, Stud.I.C.E.
Lycett, Trevor, Stud.I.C.E.
McCreath, George Forbes, B.Sc. (*Glasgow*), Stud.I.C.E.
McEwan, Ian, Stud.I.C.E.
McIntosh, Angus Lindsay, Stud.I.C.E.
Mackenzie, Finlay Peacock, B.Sc. (*Glasgow*), Stud.I.C.E.
Mckenzie, Roderick McDonald, B.Sc. (*Glasgow*).
Mackey, James Patrick, B.Sc.(Eng.) (*London*), Stud.I.C.E.
McKiddie, David Michael, B.Sc. (*St Andrews*), Stud.I.C.E.
MacLeod, John.
McNamara, Robert Graham, B.E. (*Queensland*).
McQueen, Andrew Milroy, B.Sc. (*Aberdeen*).
Manning, John Maurice, B.Sc.(Eng.) (*London*).
Manzoni, Michael Victor, B.A. (*Cantab.*).
Marsden, Antony Ernest, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Martin, Redvers, Stud.I.C.E.
Martinus, Oswald Leslie, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Mason, Peter James, Stud.I.C.E.
Masojada, Miloslaw Edmund, B.Sc. (*Natal*), Stud.I.C.E.
Mattock, Anthony Frederick George, B.A. (*Cantab.*)
Mendis, Balapuwadugе Closis Harold, B.Sc.(Eng.) (*London*).
Mills, Harold Ball, B.Sc. (*Cape Town*).
Mollison, Alexander Robertson, B.Sc. (*St Andrews*).
Mondal, Amir Ali, B.E. (*Calcutta*).
Monk, Peter Alan, Stud.I.C.E.
Montgomery, Rex Walton, B.E. (*New Zealand*).
Morris, James Victor, Stud.I.C.E.
Morris, Maurice Colin, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Morris, Michael Edward, Stud.I.C.E.
Morriss, Christopher John, B.Sc. Eng.) (*London*), Stud.I.C.E.
Morton, Robert Russell, Stud.I.C.E.
Mosley, Peter, Stud.I.C.E.
Müller, Eckhard François, B.Sc. (*Cape Town*), Stud.I.C.E.
Neave, George Neilson, B.Sc. (*Glasgow*), Stud.I.C.E.
Ng Peng Khoon, B.Sc. (*Illinois*).
Nissanga, Don Bermulan Prematilaka, B.Sc.(Eng.) (*London*).
Norman, Frank Daubeney, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Norman, John Michael, B.Sc.(Eng.) (*London*), Stud.I.C.E.
O'Malley, Augustine Anthony, B.E. (*National*).
Onions, Allan, Stud.I.C.E.
Opara, Donatus Onyejela, B.Sc.(Eng.) (*London*).
Ott, Anatole Konstantin Hermann.
Owen, Gwilym Trefor, B.Sc. (*Wales*), Stud.I.C.E.
Page, Derek, B.Sc.(Eng.) (*London*).
Park, John Anthony, B.Sc. (*Manchester*). Stud.I.C.E.
Parkes, Douglas Brian, B.A. (*Cantab.*), Stud.I.C.E.
Parks, Maurice, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Parnell, Brian, B.Sc.(Eng.) (*London*).
Parsons, John Edward Blackwell, Stud.I.C.E.
Payne, John Stuart, Stud.I.C.E.
Pearce, David John, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Pearlstone, Bernard.
Peet, Ronald, Stud.I.C.E.
Phelan, Richard Geoffrey, Stud. I.C.E.
Pond, Roy Victor James, Stud.I.C.E.
Ponrajah, Anthony James Princely, B.Sc.(Eng.) (*London*).
Pope, James Arthur, Stud.I.C.E.
Price, Dennis Clifford, B.Sc. (*Wales*), Stud.I.C.E.
Price, Derek Arthur, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Quin, Richard Wyndham, B.Sc. (*Bristol*).
Quinn, James Ignatius, B.E. (*National*).
Raikes, Roger Melville Taunton, B.A. (*Oxon*), Stud.I.C.E.
Read, John, B.Sc. (*Bristol*), Stud.I.C.E.
Rees, Donald Bowen, B.Sc. (*Wales*).
Richards, John Reginald, Stud.I.C.E.

Richardson, John Keith, B.Sc. (*Leeds*), Stud.I.C.E.
Robjohns, Ronald Henry, B.Sc. (*Bristol*).
Roe, Derek Frederick, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Rolfe, Frank Valentine, Stud.I.C.E.
Rose, John Edward, Stud.I.C.E.
Rosten, Leonard, B.Sc.(Eng.) (*London*).
Russell, Alexander Forbes, Stud. I.C.E.
Salloway, Edward, B.Sc.(Eng.) (*London*).
Samuel, Peter, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Sanders, Peter John, Stud.I.C.E.
Scholnick, Gabriel, B.Sc. (*Cape Town*), Stud.I.C.E.
Scott, Arthur, Stud.I.C.E.
Shihabi, Faruq Zuhair, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Simpson, David, B.Sc. (*Manchester*).
Sivaloganathan, Kathirithamby, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Sivasubramaniam, Appudurai, B.Sc. Tech. (*Manchester*), Stud.I.C.E.
Sivasubramaniam, Sinnadurai, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Skeldon, Peter John, Stud.I.C.E.
Skinner, Ronald Alister, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Smith, David William, Stud.I.C.E.
Somasunderam, Selladurai, B.Sc. (Eng.) (*London*).
Southgate, Raymond Thomas, Stud. I.C.E.
Spearing, George David, B.Eng. (*Sheffield*), Stud.I.C.E.
Spillard, Benjamin Charles, B.Sc. (*Belfast*), Stud.I.C.E.
Stapleton, Geoffrey, Stud.I.C.E.
Staveley, Peter William, B.Sc. (*Natal*).
Steggall, Malcolm Ronald, Stud.I.C.E.
Stewart, Patrick Huston, B.A., B.A.I. (*Dublin*).
Stockdale, John Kenneth, Stud.I.C.E.
Sullivan, Richard Arthur, B.Sc. (*Cape Town*), Stud.I.C.E.
Swindells, Trevor Davis, Stud.I.C.E.
Taylor, Gordon William Herbert, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Taylor, Peter, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Taylor, Raymond William, Stud.I.C.E.
Tew, Eric.
Thompson, Raymond Francis, Stud. I.C.E.
Thorburn, Samuel, Stud.I.C.E.
Toft, Derek Harold, Stud.I.C.E.
Toner, William Anthony, B.Sc. (*Belfast*), Stud.I.C.E.
Torrance, Kenneth Charles, B.Sc. (*London*).
Turner, Frank Royle, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Tyrrell, Norman Robert Steele, B.E. (*New Zealand*).
Upton, Charles Haynes, B.E. (*Sydney*).
Vaswani, Harkrishin Pahlajrai.
Visvesvaraya, Hosagrahar Chandrasekharaiya, Stud.I.C.E.
Vulliamy, Patrick David, B.A. (*Cantab.*).
Wade, Donald Hugh, B.Sc. (*Birmingham*).
Wainwright, David Keith, Stud.I.C.E.
Ward, Alfred Neville, B.Sc.(Eng.) (*London*).
Warren, Alan Arthur, B.Sc.(Eng.) (*London*).
Watson, Hugh Seymour Pennefather, B.Sc. (*Bristol*).
Watt, James, B.Sc. (*Aberdeen*).
Watts, Geoffrey Edward, Stud.I.C.E.
Webberley, John, Stud.I.C.E.
West, Anthony Richard, B.Sc. (*Durham*).
West, Robert Ernest, Stud.I.C.E.
Whateley, Reginald George, B.Sc. (Eng.) (*London*).
Wilkin, Frank Thomas, B.Sc. (*Birmingham*).
Williams, Fred Trevor, B.Eng. (*Liverpool*), Stud.I.C.E.
Williams, Peter Magnus, B.Eng. (*Liverpool*)
Wilson, Derek Lonsdale, B.A., B.A.I. (*Dublin*).
Wilson, Richard Dennis, B.Sc. (*Cape Town*).
Winders, John David, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Withers, David Henry, Stud.I.C.E.
Woolley, David Arthur, Stud.I.C.E.
Wynne, John Lloyd, Stud.I.C.E.
Yaxley, Denis Warrington, Stud. I.C.E.
Young, Ernest Frank, B.Sc.(Eng.) (*London*).
Young, William Mould, B.Sc. (*Durham*).

and had admitted as

Students

Akemu, Paul Wilson Omagbemi Onotchemuwada.
Armour, Carlyon Wallace.
Barnes, Michael Frederick.
Baty, Michael Frank.
Beaton, Harold Campbell.
Beddow, Joseph Derek Harvey.
Bender, Colin Clive.
Berry, Michael Frederick.
Bewsey, Ronald Walter.
Bickerdike, John.
Black, Ronald.
Blake, Ronald James.
Borders, Clifford John.
Bovington, Alan Robert.
Brasher, Justin Conrad.
Brayshaw, Allan Roy.
Brent, Noel John.
Brinham, Donald Lewis.
Brougham, Geoffrey Graham.
Brown, William Barnett.
Castle, Edwin John Lawrence.
Chase, John Trevor.
Chee Keng Yam.
Clark, Kenneth Victor.
Clark, Richard Andrew.
Clarke, Donald.
Clarke, Roger.
Clunas, John Michael Tudor.
Cooper, Brian.
Cooper, John Holt.
Cooper, Michael Russell.
Coulson, Rodney Richard.
Crawford, Duncan McCallum.
Crisall, Christopher James Philip.
Curley, Patrick Vincent.
Dewar, Robert.
Drake, Frederick Colin.
Dickerson, Maurice Woodruff.
Dunn, Sidney Alfred.
Dyer, Robert Hugh.
Edwards, Richard William.
Elliott, Keith Stuart.
Evans, Arthur Thomas.
Evans, Richard Anthony.
Fawcett, Brian.
Forder, John William.
Fox, Clive.
Gerryts, Abraham de Villiers.
Gibson, Ulric Philbert McKell.
Gittens, George Leopold.
Glaister, Harold Peter Dilworth.
Goodman, David Frederick.
Grant, David William Lawrence.
Greening, Trevor Guy.
Greenwood, David Gerald.
Grierson, James Halyburton.
Grixoni, Donald Cyril Francis.
Hall, Ronald.
Hamilton, Ian.
Hamilton, John McFarlane.
Harper, John Brian.
Harris, Glynne James.
Hayes, Brian Wilfred.
Hayes, Christopher Edwin.
Head, Kenneth Charles.
Heath, Michael Walter.
Hodges, John Henry.
Holdsworth, John Edward.
Holloway, John Edward.
Horton, John Philip.
Hughes, William Bolt.
Human, Archibald.
Hunt, Dennis William.
Hunt, John.
Idowu, Charles Olatunji.
Ireland, Benjamin.
Jackson, Leslie Henry.
James, David Harry.
Jennings, Duncan Richard.
Joiner, Ian William.
Jordan, Dennis Roderick.
Jubb, Edward Michael.
Kearns, Michael Edward.
Kelly, Brian Harold.
Kempley, Terence Thomas Alfred.
Kenward, Ian Joseph.
King, John Audus.
Kleynhans, Ernest John.
Knowles, Richard William.
Kowalski, Tadeusz Gabriel.
Kühn, Heinrich August.
Lancaster, Anthony John.
Lander, Raymond.
Leighton, Neil Harry.
Letman, John Albert.
Luffman, Kenneth William.
Macdonald, Keith Edward.
McClurg, Desmond Robert.
McDowell, Alexander Samuel.
MacGregor, Michael David.
Mallory, Keith.
Mangan, Anthony Joseph.
Mangat, Harcharan Singh.
Mansfield, Philip.
Marchetti, Guido Thomas.
Martin, William Frederick.
Miller, David Ian.
Mitchell, Christopher.
Mitchell, Colin William.
Mitchell, Hugh Miller.
Montgomery, Hugh.
Morley, John Howard.
Morrish, James Francis.

Muir, John Kerr.
Munnery, Walter Frank Owen.
Neale, Michael Albert.
Nelson, Raymond Pat.
Nicholas, Roy Edward.
Nisbet, James.
Osibamowo, Julius Olatunji.
Padmanathan, Thambyayah.
Padfield, Anthony David Doel.
Parnell, George Edward Ronald.
Peckham, Donald Francis.
Pepper, Peter John.
Perera, Willoraarachchige Srian-anda Dharmabandu.
Pet, John Mollett.
Pointer, Stuart.
Poologasoundranayagam, Kandasamy.
Pownall, Thomas.
Preller, Harry Michael.
Prior, Colin Reginald.
Ramsbottom, James.
Rawlinson, Derrick.
Raybould, Howard Keith.
Redmill, David Michael.
Rizzo, Noel Vincent George.
Robbins, John Bernard.
Roberts, Patrick David.
Robinson, David Carlton.
Robinson, Richard Albert.
Rowlands, Hugh David.
Sadler, William Ewart.
Scanes, Robert Leonard.
Shaw, Geoffrey Frank.
Simpson, Alan George.
Sims, Roy Blackmore.
Skellern, William.
Slaney, John Philip.
Smith, Fraser Kennedy.
Smith, Keith Francis Sedgwick.
Sparks, Abyn Dereck Walsh.
Stephens, Robert Louis.
Stephenson, Geoffrey Owen.
Stimson, John Rodney.
Stower, Timothy.
Sturgess, John Harry Douglas.
Sugden, Alan.
Tendolkar, Prabhakar Shamrao, B.E. (*Bombay*).
Thei, Arthur Abraham.
Thomas, Robert.
Thorburn, John Quarrell.
Thorn, John Haddon.
Tompkins, David Edward Alfred.
Underwood, Brian Gordon.
Vink, Pieter.
Walker, Allan.
Warner, Donald Edwin.
Webb, William James.
Weigh, Colin Ferguson.
Weldon, Gerald Duncan.
Wells, Dennis Crosbie.
West, Brian Reginald.
Weston, Francis Scott.
Wilkinson, Peter Derek.
Will, Alexander Ian.
Wilson, George.
Wilson, Harold Charles.
Wingfield, Ian Roland.
Wong Yui-Cheong.
Wood, Anthony Knowles.
Wrathall, Gerald Leslie.
Young, Colin.

The Secretary announced the awards which had been made by the Council for Papers read at meetings in Session 1953–54. Full details of these and other awards are given on p. 29.

A number of the recipients were in attendance and the President made the following presentations :—

Certificates of the award of Telford Premiums to Major-General G. N. Tuck, and Messrs Bryan Donkin and C. G. Carrothers.
A Certificate of the award of a Crampton Prize to Mr J. I. Campbell.
Certificates of the award of Telford Premiums to Sir Charles Westlake, Mr T. A. L. Paton, and Dr H. E. Hurst.
A Certificate of the award of a Trevithick Premium to Mr E. A. G. Johnson.
Certificates of the award of Telford Premiums to Messrs J. T. Williams and R. le G. Hetherington.

Certificates of the award of a Webb Prize to Messrs I. M. Campbell and N. J. Nichols.
A Coopers Hill War Memorial Medal and Certificate to Mr J. A. Neill.
Certificates of the award of Telford Premiums to Messrs T. J. Upstone and W. H. Cardno.
Certificates of the award of a Manby Premium to Messrs O. A. Kerensky and K. E. Hyatt.
Certificates of the award of Telford Premiums to Messrs H. D. Morgan, C. K. Haswell, Dr T. P. O'Sullivan, Professor A. L. L. Baker, and Mr Jack Duvivier.
A James Forrest Medal, a James Prescott Joule Medal, and a Certificate of the award of a Miller Prize to Mr D. H. Kent.
An Institution Medal to Mr J. C. Noel.

The Retiring President, introducing the new President to the meeting, said that it gave him great pleasure to do so. Mr Watson was very well known on account of all the work which he had done for many years past for the Institution. Nineteen years ago his father had been elected President of the Institution, and it must give Mr Watson added satisfaction to occupy the Chair once filled by his father.

He then requested the new President to take the Chair.

Mr David Mowat Watson, B.Sc., then took the Chair as President and called on Sir Arthur Whitaker to move a resolution.

Sir Arthur Whitaker, Vice-President, moved the following resolution:

"That the members present at this Meeting desire, on behalf of themselves and others, to record their high appreciation of the services rendered to the Institution by Mr Shepherd-Barron during his term of office as President."

It was well known, he said, that the duties of the President were arduous, but during the past year they had been exceptionally so, since there had been a change of Secretary and, owing to the unfortunate death of Mr Graham Clark, an interregnum. The Institution was fortunate that during that very difficult period Mr Shepherd-Barron had been present, with his very practical mind, to guide its affairs.

One of Mr Shepherd-Barron's leading good qualities was that he was not hidebound by tradition. The Institution was permeated by tradition. He was one of those rare mortals who, though demolishing tradition, also created it, and one or two of the things which he had done during his term of office would probably become traditional in the years to come. One of these of perhaps special interest to the younger members—although it was said that the Members of Council occupied the floor—had been that

at the Conversazione this year the Great Hall had been used for the first dance ever held there. That was due to Mr Shepherd-Barron.

Mr Arthur Floyd, who seconded the motion, said that it was impossible to over-state the debt which the Institution owed to the retiring President, and therefore it was a very great privilege to second the vote of thanks and to reinforce and support what Sir Arthur Whitaker had so ably said. It would be readily accepted that when the Institution conferred the honour of President on one of its members the Institution itself was not unaware of the distinction thus bestowed, but Mr Floyd thought he would be right in saying that when the President undertook his duties he was not very long in putting the balance of that distinction very much on the other side, and very soon became aware of the exacting nature of the duties which the Institution had asked him to undertake.

Mr Shepherd-Barron had followed in the line of succession of former Presidents who had been no less distinguished than himself. Those who had seen him in action knew very well that he had not spared himself in the service of the Institution. In leading in a most outstanding way the affairs of the Institution he had impressed his personality on them all. He had that amazing combination of gentleness and firmness which was so desirable in anyone holding such a high office and which was the hallmark of a great personality. The Institution, in conferring a distinction on him, had conferred a distinction on itself, and Mr Floyd was certain that when the motion which he was seconding was put to the meeting the members present would make that fact abundantly clear.

PRESIDENTIAL ADDRESS OF

David Mowat Watson, B.Sc.

President, 1954–55

In electing me to be your President you confer upon me the highest honour which this great Institution has to bestow. My gratitude to you cannot readily be expressed, but it is with pride that I thank you now, and in humility that I acknowledge my duty to apply such talents as I possess to your service that I may endeavour to bring credit to you. The knowledge that I am embarking on an onerous service does not deter me from my desire to serve you.

Nineteen years ago my father stood where I now stand and I have at least this in common with him that I have the welfare and the honour of our profession at heart. I am proud to think that I am thought worthy to bear my share in this great work and to have my name recorded with his.

No one could take up these duties without giving some thought to the great engineers who have occupied this Chair in the past. Eighty-nine names confront me, names of men who have brought distinction to this profession, names of men who have guided the affairs of the Institution and played their part in its evolution from the days of its foundation to the present time. One hundred and thirty-four years have elapsed since our first President took office, and our profession might be said to have been established.

That is now history and engineers do not as a rule dwell in the past and are seldom sentimental about their profession. Their outlook on life and its problems is generally realistic, their attitude progressive. Their time is usually too fully occupied to allow them to delve into the past and take pleasure in it and, for that very reason, they tend to ignore the genesis of their own being, the aims and ideals of their predecessors, and the directing force which inspired them. That our heritage in the Institution is a great one we do not for a moment doubt or deny, but we take little thought of the responsibility which is ours by inheritance. Who knows what our profession would be like today if the early idealists had held different views ? And are we loyal to those views ? Edmund Burke once said, " People will not look forward to posterity, who never look backward to their ancestors." Not only the opportunities we grasp but the opportunities we miss will be history in the future.

For a few minutes this evening let us refresh our minds on some of the conditions of the past, and maybe contrast them with the conditions of today when our difficulties are so very different. Perhaps we may even gain inspiration by thinking sometimes of the rapid strides the old pioneers

made and the originality of thought they brought to bear on what to them were new problems. They were bold in their conceptions; are we bold enough? Are we too ready to accept the restrictions imposed on us by modern conditions? Are we so hedged around with stereotyped ideas, Governmental regulations, standards, and "usual practice," and even thoughts for the safety of our own positions, and provision for our old age, that we are afraid to indulge our own individuality?

John Smeaton (1724–1792), often called "The Father of Civil Engineering," probably deserves pride of place as the founder of our profession. He it was who not only brought to it technical education, which resulted in scientific application of the crafts, but inspired a professional spirit amongst his followers and successors—a spirit which may indeed have resulted indirectly in the founding of our Institution. These followers discovered a means of expressing their corporate spirit and seriousness of purpose when they founded the Institution. In the past we have not sufficiently honoured them.

How many of us remember the story of the origin of the Institution and that it existed for 2 years before the first President, Thomas Telford, was invited [1] "to patronize the Institution by taking on himself the office of President of the same?" After all these years we are at last making tardy recognition of Henry Robinson Palmer as our founder. He it was who deserved pride of place because it was he who first brought together in a London coffee house [1] a number of enthusiastic young engineers on Christmas Eve, 1817. He read a Paper on that occasion and so inspired his audience with the idea of association, that only 9 days elapsed before they met again and resolved ". . . that a Society be formed consisting of persons studying the profession of a Civil Engineer." Four evenings later they again met to elect two Chairmen—Field and Palmer—and a Secretary—Jones; and yet again they met only 7 days later on the 13th January, 1818, to adopt the title we use to this day—the Institution of Civil Engineers.

In that short space of 20 days those young men held no less than four meetings and showed in that way, even if in no other, that they felt the need of something in their professional lives. What it was they needed we are not told, but we must assume from the results achieved that they felt the need of association and cohesion of civil engineers, not unification to promote better conditions for themselves but to facilitate their own learning, and to foster pride in their own profession and the conduct of those engaged in it.

That story is on record in the Institution and known to you all, but I summarize it here because it was the foundation of our education, our training, our facilities for learning from our colleagues, the literature of our Institution, our recognition of our fellows, our desire to act in accordance

[1] The references are given on p. 26.

with the highest ethical codes of professional conduct, and above all our professional pride.

When our third President, Sir John Rennie, assumed office in 1845 he delivered a brief address [2] in memorable terms, and I make no apology for quoting it at some length, because it expresses more eloquently than I could hope to do how much this noble profession of ours calls for originality and inspiration. He finished that address by saying—

" When we look around us and see the vast strides which our profession is making on every side, and the deservedly high place it holds in public estimation, we cannot but feel justly proud ; for without the slightest disparagement of the pursuits or studies of other professions, I may confidently ask, where can we find nobler or more elevated pursuits than our own ; whether it be to interpose a barrier against the raging ocean, and provide an asylum for our fleets ; or to form a railway, and by means of that wonderful machine—the locomotive engine—to bring nations together, annihilating, as it were, both space and time ; or to construct the mighty steam vessel, which alike regardless of winds or waves, urges onwards its resistless course ; or to curb and bring within proper bounds the impestuous torrent, converting its otherwise destructive waters to our use and benefit, whether for navigation, trade, or domestic comfort ; or the drainage of the unwholesome marsh, and converting it into fields of waving corn ; or illuminating our cities with gas, changing, as it were, night into day ; or the fabrication of machinery of endless form and ingenuity, by means of which every article, which can tend to man's comfort and luxury, can be produced in the greatest perfection, at the smallest cost ; or to recover from the bowels of the earth nature's exhaustless treasures, and forming and preparing them to our use. In fact we may almost say, that there is nothing in the whole range of the material world, which does not come under our observations, or where the skill, and science of the engineer is not required, in a greater or less degree, to render the bounties of Providence subservient to the good of mankind.

" With such splendid prospects before us, we have every inducement to stimulate our zeal, and to press forward in the career of improvement. Thanks to our illustrious predecessors, Smeaton, Watt, Brindley, Jessop, Huddart, Chapman, Telford (and I may with pride add to these the name of Rennie), and to our present distinguished members, much has been done, but still more remains to be done. With this excellent Institution (which has already achieved so much good) as our rallying point, in which all our energies should be centred, there is no doubt but that our exertions will be crowned with success.

" Our first object should be to render our meetings interesting and instructive, by the production of good papers on every subject, connected, directly or indirectly, with the profession. Let us all contribute to the best of our ability and opportunity ; those amongst us who may be too much occupied with the active duties of business to become authors

themselves, may still be of the most essential service to the Institution, by communicating, in verbal discussion, those facts, as the result of their experience which they have not time to record; and by directing their assistants to note and describe all interesting points, which are constantly occurring every day, more or less, in the routine of professional practice and which are too often lost to the world and the profession, for want of being recorded.

"Let the junior members of the profession keep a regular journal of every thing which passes under their observation, and endeavour to classify the facts in such a manner, that they may be enabled to deduce general principles from them, which may be applicable under similar circumstances; and let our non-professional and amateur members assist us by their liberality and support, by sending us original communications, books for the library, or models for the collections; but above all, let us be true to ourselves, and banish all other feelings except those of unanimity, harmony, and kindness.

"By these means this most excellent Institution will flourish, and the great profession which we follow will, if possible, become still more elevated in public opinion; and we shall enjoy the proud satisfaction of knowing, that by our united exertions the grand objects of the advancement of civilization, and the happiness of mankind will be promoted."

In those inspiring words Sir John Rennie described our profession, urged us all to greater endeavour, and showed how we may all augment the prestige of our Institution, and thereby benefit man.

It is recorded in the Annual Report on the Session 1837 printed in the very first volume of the Minutes of Proceedings, that the Council "believe that the time is fast approaching, when a general conviction of the advantage which must result from the periodical meetings of the members of the same profession, will induce all scientific men to unite in advancing the great objects which the original projectors of the Institution had in view, and which the Council has constantly laboured to promote." In the same report two measures of the size of the Institution were given:—the one was a total membership of 252 comprising only 47 Ordinary Members, 93 Corresponding Members—the equivalent of our Associate Members—98 Associates and 14 Honorary Members; the other is taken from the published accounts for the previous year and shows the total expenditure for the year to have been £851 11*s.* 4*d.* Today our comparable figures are about 13,000 corporate members and about 7,000 graduates and students making a total of more than 20,000 and an annual expenditure of about £95,000.

It would be true to say that our aims and objects are the same today as they were in 1837, but the scope for prosecuting them is much wider and the work involved in their achievement is greater—perhaps infinitely greater. Our membership and organization are incomparably larger, although that was to be expected, as the growth of civil engineering works

demanded more civil engineers, but those engineers would not have become members of the Institution had not the Institution been wisely and properly controlled. On that, and on insistence on suitable education and training, rather than on the growth of our membership should we feel a sense of satisfaction in our status amongst the learned Societies.

Although Sir John Rennie spoke [2] in 1845 of the "deservedly high place our profession holds in public estimation" it is doubtful if the public of today are given the opportunity of forming an estimate of the place held by the civil engineer who is too often content to be just a back-room boy, of whom the public has never heard. His work is taken for granted; he is too modest and does not seek recognition of the essential part he plays in inspiring, planning, and working out all the practical details which eventually give place to the actual direction of the great sources of power in nature to the use and convenience of man. That we should blatantly advertize ourselves is an objectionable and an improper thought, but our Institution has given us some lead in how to make known with dignity some of our works. There is still need, however, for clear thinking to abolish all foolish assumption of modesty, untimely silence, and cloistered indifference.

The public do not know the heavy responsibility that rests on the engineer in spending public money on capital works; they do not realize that his lengthy processes of comparing many ways of doing even a simple job, assessing the life as well as the annual costs, is by no means a mechanical process. It is his experience and judgement which prompt him to recommend only one of the many schemes he has considered. Thereby he assumes the real, if not the nominal, responsibility.

Just as charity begins at home, so should modest aggrandizement of our own importance begin within our own ranks. There are many Associate Members who qualify for Membership but do not apply for it. It is an honour to hold the highest rank of one's profession and the prestige of both the individual and the Institution is enhanced by transfers. If we ourselves fail to show justifiable pride in our own status, can we wonder if the public fail to recognize our qualifications?

In the early days the term "civil engineer" was an all-embracing designation for all non-military engineers and the scope of their activities was the wide definition of our Charter. The very magnitude of the subject led rapidly to the need for specialization; nowadays, the accumulation of experience and research with their considerable bibliography, combined with better technical education, has inevitably necessitated greater and greater specialization as time has marched on. For convenience the term "civil engineer" has become more restricted in its meaning since our brothers in mechanical and electrical engineering founded their own Institutions, but fundamentally, and in fact, this Institution remains available to all practising engineers who comply with our membership conditions. In more recent times confusion has been caused in the minds

of the public by the formation of engineering institutions devoted in their interests only to departments of civil engineering—departments which are included in civil engineering as judged by even the most restrictive definitions of today. To make confusion worse confounded the very word "engineer" has become so wide in its use and so loosely applied that it is doubtful if it has any value left. In 1922 the Institution obtained a supplemental charter empowering all its corporate members to call themselves "Chartered Civil Engineers" and this had at least the merit of enabling the professional engineer to describe himself with the minimum of confusion. The term is perhaps not ideal but it is the best available to us and its more common use would result in education of the public.

In the Annual Report on the Session, 1838, Telford was quoted [3] as having once said that "Judicious regulations are absolutely necessary in all societies, but I trust that in this the good sense of the members will always feel that manners and moral feeling are superior to written laws." We civil engineers claim that our education is wide and embraces many technical studies and subjects; that our activities bring us into contact with the labourer, the craftsman, and the academician, the administrator, and members of other professions; that our experience teaches us to be just, to weigh our facts, to avoid hasty decisions, to look at all points of view. Surely, if our claims are justified, we can govern our own behaviour, treat our fellows fairly, and comport ourselves with propriety in our personal and professional capacities. If we can, if we are all of one thought, if we all conform to the rulings of our own professional colleagues, then we can truly say that "manners and good feeling are superior to written laws." In our Institution we have written laws because every community must have a government of some kind, but the test of our behaviour is the frequency of infringement of those laws. The Council has a committee which is concerned only with professional conduct and contraventions of the Institution's written laws of conduct. I have served on that committee for a long time now and can say with pride that it is by far the lightest-worked committee of the Council. The alleged delinquencies are few, and the committee seldom has to meet. Let us take pride in that fact and its implication; let us continue to be influenced by our loyalty to our own great profession, and its etiquette.

With all his wide training and extensive experience of men and the world, the engineer seldom enters the realm of pure administration. Yet he has so many of the necessary attributes and qualifications; he must know men from the labourer upwards; he must learn which of them to trust, and which to promote to greater responsibility; his whole outlook has been directed to just and fair dealings; his experience has taught him how to delegate responsibility to others but to continue to direct their endeavours to suit the combined efforts of a whole team of workers, always with the main end in view; his scientific education is broad enough to enable him to appreciate the results, if not the difficulties, of

scientists, and his experience has taught him to handle men to get the best out of them, from the highest to the lowest. Senior engineers devote most of their time to administration, but only a few make their unique experience of human nature and the wide range of engineering economics available in administrative posts.

Comparison of our difficulties today with the difficulties experienced by the engineers of the early formative period in the history of our profession would be unprofitable, even if possible. Nevertheless, when we are tempted to think unhappily of our handicaps, frustrations, and inability to get on fast enough, it is not only a comfort to think of the difficulties of olden times but a stimulus to our fighting spirit. I think there was more prejudice and doubt for the old engineers to overcome before being allowed to undertake a project; I think they had greater trouble in persuading employers or clients that they had the requisite ability and experience to command respect. Nevertheless, there was a prevalent spirit of adventure in the world which could be invoked by men of such strong personality as some of the early engineers. At times their office problems must have been acute just as ours are. Where could they get assistant engineers to staff the office in the days when few men had both scientific knowledge and experience of the constructional crafts? How much time would be available to direct the office staff whilst recruiting a labour force and suitable men to take charge of it many miles away? Keeping in touch with those works involved travelling, which was incomparably slower. To read of the numerous works of Telford in all parts of Great Britain and to learn that he was in London for the Parliamentary Session and perhaps touring the Highlands on horseback to carry out comprehensive inspections of, say, the 60 miles of the Caledonian Canal, or all the harbours of the north and east of Scotland in the same year makes one wonder how he found time even to know what was going on on his own works. A moment's thought only is needed to see in such activity, without railways, roads, motor cars, aircraft, or quick postal service, telegraphs, and telephones, a multitude of tribulations we are mercifully not required to overcome. Speedier ways of travel and communication, better education and technological knowledge, as well as a fundamental development of carrying out the constructional work by contract, all had an important influence on the work of the engineer and his capacity for more and better work.

The public works contractor of today developed from this environment and grew from the modest beginnings we would today call sub-contracting. The engineer began to delegate to him the construction of the designed work, until now he hands over to him the whole of the construction. Into the contractor's hands is put a complete delineation and description of what is wanted, setting out the conditions under which it is to be done, and the engineer is then free to concern himself primarily with the permanent outcome of the operations, whilst the contractor must himself look to the economics of the construction. As technological education

has advanced, so the contractor as well as the professional engineer has benefited by collecting about him trained engineers.

There is today a working partnership between the professional engineer and the contractor which is a relationship of which we can be proud. At times our points of view differ and in our enthusiasm we are ready to miscall one another with alacrity, but seldom with sincerity; only occasionally do we differ seriously and have recourse to arbitration or the Law Courts. Our differences are resolved amicably enough and justice is done to our complaints of one another. Confidence between the two bodies has been established. To preserve it, however, the engineer throughout the administration of a contract between the employer for whom he is acting, either as a full-time salaried officer or as a consultant, and a contractor must act with scrupulous fairness whether the result is for or against his own employer. His state of independence to judge fairly between employer and contractor must be unassailable, and any attempts to remove from his shoulders even a part of that great and honourable responsibility should be resisted strenuously. The intervention of a third party would, I believe, wreck these conspicuously successful relations at the ultimate price of all public works. Too many of our public authorities are tending to bring into the administration of the contract other officials in addition to the engineer. That is deplorable and can only do harm in the long run, by destroying confidence, introducing unnecessary uncertainties, delaying settlements, and eventually raising costs. Obligation to be loyal to our heritage is heavy on us all but the duty to resist this interference is squarely set on the shoulders of the professional engineer.

Just as the public works contractor of today has developed from the technically unqualified employer of labour, so have his tools changed beyond recognition from pick, shovel, and wheelbarrow to mechanically operated machines of astonishing mobility, size, and purpose. Their economic employment to lessen the cost of all operations is an important factor affecting the capital cost of all public works, but it is not possible for the engineer to take full advantage of it when he is designing, because he is then unaware of the identity of the successful tenderer. Had he been aware of it he might have adopted a design differing to some large or small extent. The desirability of public tendering is too well known to need comment, but is it beyond our ingenuity to evolve some system whereby the engineer can take full advantage of the facilities which will be available during construction and of the experience of the men who know best how to handle their own plant to get the best out of it? Looking into the future it seems that the imperfectly informed may tend to favour "all-in bids" by contractors tendering on design as well as construction, and if so, they will thereby fall a prey to all the serious evils of competitive design. Overseas work affords some evidence that this is not an unfounded fear. That the merit of a design cannot be judged by cost is known by us so well that it is not worthy of mention, but, astonishing

as it may be, it is not known by very many bodies who ought to know; and we should help them to understand.

So far I have thought of our profession in its widest application but I turn now to the more limited sphere in which I have so long been engaged. If this has absorbed my attention over a much longer period than it has remunerated my professional labours the reason can be found in my unusually youthful opportunities and the atmosphere of parental discourse, discussion, and objective training in which I grew up. Although no doubt too young to appreciate all the questions involved, I was aware of many of the bold pioneering steps taken by my father in the field of public health engineering, and am proud to think of the unusually close relationship of thought and discussion which always existed between us.

Tonight, however, I want to remind you of the small beginnings of the public health engineer as reflected in the Proceedings of our own Institution. A mild interest in public health can be said to date back 200 years but it was not until the time of Edwin Chadwick (1800–1890) that a substantial weight of opinion began to demand the removal of sewage matter from towns, and the use of public money, spread over a period of time, to pay for the sewers. Only gradually was this accepted as a necessity in the creation of healthy living conditions in towns, but progress in this respect may have been caused by the needs of the industrial age and stimulated by the more charitable outlook born of the religious revival. This country then led the world in public health work.

As recently as the 1830's and 1840's the discussions which were recorded in the Proceedings indicated that many members took part even though lacking in experience of the drainage of towns. That was natural enough because few specialized and all were, therefore, willing to give serious consideration to all engineering problems. Public health engineering was a new subject and engineers were apparently struggling to find the best approach to it before settling down to engineering details of how best to give effect to ideas. For instance, in 1839 Jones, in writing,[4] of "Sewage of the City of Westminster," stated that the Commissioners of Sewers of that time were guided by the Act of Henry VIII—for the most part applicable to fen drainage. These Commissioners were not ". . . invested with powers enabling them to originate new lines of sewers, but being confined to improving those that exist, and controlling the construction of new ones." He wrote, "A large portion of Westminster is below the level of high water, and the drainage of buildings being optional on the part of the builder, there consequently exist insulated houses and districts of loathsome filth, for want of sufficient compulsory powers on the part of the Commissioners." Even in 1852 we read of ". . . miles of sewers in the metropolis which do not receive the drainage of one-tenth of the houses of the district," a comment which apparently caused no surprise, and shocked no one.

More light is thrown on this subject in a Paper by Robert (later Sir

Robert) Rawlinson, in 1852, and in the ensuing discussion which ran to portions of four evenings. In that Paper, which was entitled "On the Drainage of Towns," Rawlinson stated[5] that "In Paris the contents of water closets are generally excluded from the public sewers, and it is only recently, that the law has been repealed in Liverpool, forbidding the turning of water closet refuse into the sewers." During the discussion, however, Haywood stated that ". . . erroneous ideas prevailed as to the condition of the Paris sewerage; formerly there was an entire prohibition of any fâecal matter going into the sewers, and that prohibition existed legally to the present time; but, by degrees, exceptions were made in favour of the prisons, the barracks, the hospitals, the markets, and other public buildings, all of which had for many years communicated directly with the sewers, which debouched in the Seine, in the middle of the City."

The Duke of Buccleuch's Royal Commission in 1844 quoted figures for water supplied to houses. In Bristol, for example, only 5,000 people were served out of 130,000; Birmingham, 8,000 houses served out of 40,000; Coventry, 300–400 houses served out of 7,200; Newcastle, only 8 per cent of the houses served, and in Nottingham there were no less than 70,000 houses without water. Even in 1861 Homersham stated[6] in a discussion in the Institution that of 13,000 houses in the City of Leicester only 700 were connected with the sewers of the town.

Such excerpts from records serve to emphasize the age of the Institution and simultaneously, though somewhat anomalously, the recent nature of the fundamental concepts of public health engineering. Fifty years or so later the public health engineer began to say that not only the contents of water closets but all liquid wastes, even from trade premises, ought to be discharged to public sewers. Only in quite recent times, however (Public Health (Drainage of Trade Premises) Act, 1937), has the manufacturer or owner of trade premises been given the right, conditionally, to get rid of his liquid trade wastes into the public sewer, but the practicability and desirability of the practice was gradually growing stronger over this longer period of time, and was unquestionably due to greater knowledge amongst engineers.

Most of the sewers of that early time were built of brick; pipe sewers, as an innovation, encountered many failures and much adverse criticism. This controversy which runs through a number of the early discussions was not just academic, it was quite clearly a major issue with many speakers. It was claimed that it was unfit for a man to crawl through a sewer and in support of that argument it was said that new legislation prevented boys from being sent up chimneys, but that more lives had been destroyed in foul sewers than were ever lost in crooked chimneys. The contrary view held, however, was that crawling through sewers was voluntary and therefore not comparable. Figures were quoted in many discussions to show that the cost of large brick sewers was actually less

than that of small pipe sewers and that, since they did not collapse, there was no replacement cost, as was necessary with broken pipe sewers.

Rawlinson in 1852 spoke[7] of sectional forms of sewers as being V-shaped, square, oval, and circular and also "partaking of every combination of these figures," and he spoke of egg-shaped sewers as small as 4 inches in diameter.

While all this controversy was raging there were simultaneously and intermingled with it, other conflicts of ideas about deposits in the sewers and the hydraulic formulae suitable for calculation of sewer sizes, but the greatest problem of all was one we have not yet completely solved; how best to treat the sewage collected, how to dispose of it without nuisance, and at the same time to utilize what is of value to the best advantage.

In the discussion following a Paper by Green[8] in 1848—a discussion which was opened by the Dean of Westminster—Lord Robert Grosvenor, M.P., spoke[9] of the desire ". . . to preserve and make beneficial use of the manure deposited in the various conduits, and instead of, as at present, pouring it into the rivers, destroying the fish, and forming shoals of foetid matter on the borders of the streams, to make use of it for agricultural purposes." Another speaker spoke of plans which had been proposed, ". . . for precipitating and deodorizing the matter of cesspools and drains, so as to convert it into a dry portable manure; . . ." He added, "If the sewage water of London were preserved and applied to agriculture, it would go far to annihilate the necessity of rates altogether." He referred to Edinburgh as having for nearly one hundred years irrigated meadow lands with town sewage.

This latter claim was, however, heavily discounted in later discussions when most uncomplimentary remarks were made about the state and smell of these meadows. Nevertheless, many towns were disposing of their sewage on land by irrigation, but apparently the primary idea was to benefit the land rather than the watercourse into which the effluent was discharged. This attitude is illustrated by remarks[10] made in 1852 by Bidder who criticized the General Board of Health and "their impracticable schemes, for the distribution of liquid manure, by pipes and mechanical means, over great extents of country, not merely adjacent to but at considerable distances from the towns intended to be drained. As an instance in point, he might mention, the proposition in 1849, to pump sewage water from London to Brentwood, a distance of 16½ miles, to an altitude of 420 ft above Trinity H.W. level, through pipes of 7 inches diameter, for the purpose of irrigating an estate, and thence to continue the same sized pipes for upwards of 50 miles further, because it would 'be easy to send the sewage on as far as Colchester, or Ipswich, that being all down hill.'"

Controversy on this question of irrigation arose at many of the meetings and the value of land with and without sewage irrigation, value of sewage as a manure, smell, and danger to health were all hotly debated, though

the need to avoid unnecessary pollution of watercourses never seemed to have been so seriously considered as it is today.

Our point of view has understandably changed. In those early days the first problem came first, namely, to remove sewage from dwelling houses and populated areas; whereas today, with this difficulty solved, we concentrate more on the resultant difficulty of conservation of river-water purity. Statistics of decrease in infectious disease, particularly water-borne disease, are used to show improvement in sanitary conditions as well as advance in medical practice, and formerly the emphasis was on those statistics; today the focus of attention is on river pollution.

That great pioneer of sanitation and public health legislation, Edwin Chadwick, is recorded as having taken part in a discussion in the Institution more than once, but on one occasion he directed attention to several fundamental points. That was in 1865, when he took part in the discussion[11] on Bazalgette's Paper on the Main Drainage of London. He advocated rational drainage areas as defined by natural not artificial boundaries, self-cleansing sewers and drains, abolition of right-angled sewer junctions, better sewer-discharge formulae, and "since he had promoted the use of pipe sewers" he advocated them though many had in the past been badly made and badly laid. But his contribution to the land treatment of sewage was the most stirring; he is quoted as having said that the soil itself is the best storage reservoir, the best deodorizer, and the cheapest; that the need was for self-cleansing sewers, quick discharge from the houses as well as from the streets; that fresh sewage—not putrid at the outfall—has no smell. He went on, "Fresh sewage is too valuable to be thrown into the river to feed fish, and should be on the fields to feed men. A poetical French writer," he continued, "Victor Hugo, had said on this subject: 'These heaps of ordure in the corners of the streets—these cart loads of manure which go jolting along the streets at night—those horrible tubs of liquid—the pestilential flood concealed beneath the pavement—do you know what it all is? It is the meadow in flower—the green herb—the wild thyme and clover—it is the abundance of game—it is the flocks of sheep and herds of cattle—it is the contented lowing of oxen in the evening—it is the scented hay and golden corn—it is the bread upon our table—it is the rich blood in our veins—it is health—it is enjoyment—it is life; so it is ordained by that mysterious creation, which is transformation on earth and transfiguration in heaven. Give all this to the great crucible, and receive back abundance. The nutrition of plants is the food of man.'"

Water-carriage of sewage was by no means readily accepted by engineers as the proper method of removal of sewage from dwellings; long-drawn out and frequently recurring are the records in some of our early Proceedings when this principle was challenged and keenly debated. It was said, with some justice, that the problem was only moved from one end of the sewer to the other where it was vastly larger and more complex in

nature owing to the large volume of water involved. But the danger to public health was unquestionably moved from the urbanized areas to the watercourses, and land treatment, long practised in many places, came into greater prominence as the only proved method of purification of the sewage. That brought in its immediate wake the question of separating the settleable solids from the liquid sewage and for years there were intensive studies and lively discussions in the Institution about methods of separation, mechanical screening, chemical precipitation, and purely physical settlement in tanks.

It seems obvious to us today that the champions of water-carriage of sewage should ultimately win the day, because we know now that by no other means can the largest amount of unclean matter be removed from a town in the shortest time. That is the engineer's duty and although the method he employs today is unchallenged he might perhaps profit by stopping occasionally to satisfy himself that he is approaching as near to the perfect state as he can. Are we entirely satisfying our conscience? Is the removal as efficient and complete as it should be? No doubt we are approaching the ideal; and the now stimulated acceptance of trade wastes, often of an obnoxious nature, into public sewers has in recent years acted as a spur to the cleaning up of industrial cities even if not directly of the actual dwellings. Do we remove and treat as much rainwater as we should? Do we pollute watercourses with the washings of many impervious areas and call it, euphemistically, "surface-water"? Discharged from a system of sewers entirely separate from our foul sewers this surface-water enjoys immunity from the stigma of the word "sewage," but we know that on occasions it is equally objectionable. Are we entirely happy about those overflows from foul sewers coming into operation when rain causes the flow in the sewer to exceed a rate glibly described as "six times the dry-weather flow"? Why six times? And is "the dry-weather flow" not in itself rather an academic term inducive of argument? Six times may be right, but are we always happy about it? Is it not an empirical figure which may be wrongly applied? We know that the Royal Commission on Sewage Disposal arrived at that figure and recommended it as the result of much evidence and earnest consideration and we respect highly the monumental work of the Royal Commission, but do we perhaps shield behind it as a habit rather than as a considered decision specifically applicable to the individual case?

Although sewage works have almost entirely superseded the old sewage farm, and occupy perhaps only a hundredth part of the area, there can be no doubt that our methods are still liable to undergo evolutionary, or even radical, change. Modern practice has been built up piecemeal over a period of many years; trial and error, research, and economics having played their important parts, but it is very marked that the scientific knowledge brought to bear on the operation as well as on the design is very materially greater today than it was in the past. It is

certain that the manager, the chemist, the biologist, and the engineer, separately or collectively, will introduce changes in practice just as they are collectively concerned in overcoming many of the operational troubles and imperfections. In sewage-purification works design, therefore, less than in most branches of civil engineering, could it be claimed that a given design would long remain up-to-date.

For example, we follow long-established custom when we say that sludge disposal is the most troublesome part of the problem of sewage treatment. We have a greater choice of method of treatment of sludge at our command today than ever before, but we are constantly teased by the fundamental question of final disposal. Can the dried sludge in its final condition be usefully applied to the land, and are economics completely reliable when we decide to dump it on land or at sea ? The cost of transportation from sewage works to agricultural land is a serious drawback to utilization on land, but money, and even labour values are fluid in a changing world. The absolute criterion is use or waste.

Mechanization of sewage works has been gradual in the past 50 years. Manual operations of many kinds are now carried out by mechanical and electrical plant. For example, tank bottoms are now swept clear of sludge by machines while the tanks are still operating, whereas the tanks were formerly emptied and the floors cleaned by men pushing rubber squeegees. But in a world of changing values economics decided in favour of the machine. None of the many processes of the modern sewage-purification works is now without its mechanical and electrical plant ; many of those processes are highly mechanized, and the value of such additional facilities has been taken advantage of to improve efficiency and control of plant. Consequently, sewage-purification plant is today complicated to the extent of requiring mechanics and electricians in increasing numbers and the management must be better informed technically than was once necessary. Process supervision, too, is of a higher technical degree, and better control of plant is assured owing to the constant vigilance and work of chemists.

Simultaneous with the gradual mechanization has been the improvement of education in Great Britain, and men employed on sewage works have become better able to appreciate the meaning of their labours and to interpret their instructions. Their knowledge of, and sympathy with, things mechanical has grown with the mechanization and maybe for this reason they have less taste for hard or dirty jobs where a machine is known to be able to do those jobs equally effectively. The result is happy enough, because where once the labour costs for those jobs were less than the costs of machinery, the reverse is so often the case today.

It is no accident that sewage works have become cleaner and tidier places during these transitions. Better education and higher living standards have led all men to develop and practise greater self-respect, and better working conditions follow. There need not now be any dirty jobs on sewage works and hence slovenliness amongst the men ; tidier

thinking can, therefore, be reasonably expected, combined with greater working efficiency.

The public mind has passed beyond the early ideas that sewage disposal can be made to show a profit; it has advanced so far as to recognize that efficiency and lack of offence are a moral as well as a legal obligation; it is slower to realize that cleanliness, dignity, and general amenity as well as spit and polish play an important, if not an essential part in the public service. There is no end to the details, the inclusion or exclusion of which, may be debated as either extravagant on the one hand or niggardly on the other. Refinements in control and more complete recording of process facts apart, the answer to such questions must in many cases depend on common sense, psychology of the public, and of the men employed, as well as on judgement born of experience. Never should it be overlooked, however, that money spent on appearance may be wise expenditure. In some parts of Great Britain there is some difficulty in getting labour for sewage works and there are probably several reasons for that, but one of them will often be found to be that the sewage works are needlessly unattractive. Sewage works presenting an unpleasant appearance are more often said to cause smell nuisances than clean and tidy works.

Too little is known about the cost of sewage purification. The service is vital to Great Britain and the cost of it, though not vital, is of first importance to the ratepayer. Prior to the 1939–1945 war the Ministry of Health published, but only for a few years, analyses of expenditure incurred by Joint Boards, County Boroughs, Non-County Boroughs, Urban, and Rural District Councils on sewage disposal. Naturally enough, however, the plant employed and the treatment given are different in all places; the population served, the volume of sewage and storm-water treated, and the trade wastes and strength of sewage all vary, so that costs, even when averaged, must be used with caution. The Institute of Municipal Treasurers and Accountants issue annually a "Return of Rates levied in the County and Non-County Boroughs and in Urban and Rural Districts," but unfortunately there is only the one rate given for both sewerage and sewage disposal. It is a serious reproach to us that a country so densely populated that abstraction of potable water from, and discharge of sewage to, our rivers is of ever-growing importance in our national life, should lack basic information about the cost of purification of one of the main discharges to the rivers.

It is not universally known that the real difficulties of sewage purification are due to discharges of liquid wastes from trade premises. Most of these wastes can be purified satisfactorily when mixed with domestic sewage but the proportions are sometimes of paramount importance. Indeed, many trade wastes, if discharged in too great concentrations, are inhibitory to biological purification and it often happens that, in spite of all careful control at the sewage works and restrictive regulations at the factories, a discharge takes place which completely upsets the purification

plant and in extreme cases renders it almost nugatory for days on end. No amount of knowledge of the best way to treat troublesome wastes avails against such varying conditions. The manager of the sewage works is helpless to control the quality of the raw sewage; his purification plant will stand up to considerable variations in duty but it has its limitations, and rapid adaptability from one duty to another is beyond man's control.

Carelessness, the human error, accident in the manufacturer's premises, or even on the public road, all threaten the efficiency of the work done by the sewage-purification plant, and it cannot be too widely known that complete protection of our rivers from such acts of the public, whether accidental or illicit, is impossible.

In their final report of 1914 the Royal Commission on Sewage Disposal recommended that sewage effluent for discharge to an inland river where the dilution factor is at least eight, be purified to a defined standard which they called the "Normal Standard." That standard is more commonly recognized than any other and has been usually accepted in the past by river authorities as the reasonable limit of purification. Nevertheless, it sometimes results in pollution, a word of unequivocal meaning. The law, though permitting discharge of sewage, prohibits pollution, so that the sewage effluent must be at least as pure as the river water. In some circumstances this is impracticable and in some it is impossible.

Local authorities must, amongst many other duties, foster industry and drain the whole town. By so doing they make a rod for their own backs because too often the result is a sewage difficult to purify to the exacting standard required by law. Where a manufacturing town is situated on a small clean river the local authority may conceivably find itself under an Order of the Court to purify its sewage to an impracticable or even impossible degree.

Industrial wastes change as scientific discovery promotes new manufactures or dictates changes in old processes, so that the wastes discharged to the sewers alter in the course of time. Some of them are notoriously difficult to treat and all depend on the proportion of domestic sewage with which they are mixed. If the industrial wastes are in themselves a serious threat to our rivers, as for example radioactive substances, or the domestic sewage is itself charged with modern detergents, the resultant purification problem may readily be beyond our present knowledge. Superimpose on this difficulty the smallness of our rivers, their increasingly heavy duty of conveying sewage effluents, and the increasing demand on them to supply water for public consumption, and the composite problem is seen to be one which cannot be resolved by harsh legislation.

It is known today that the sewage of our industrial towns can be purified far beyond the Normal Standard and it may safely be inferred that all sewage effluents could be similarly improved. The process is costly, but even so it does not always go far enough to satisfy the stringent demands of the law. Surely a country which led the world in public health engineering

should not tolerate the possibility of a local authority suffering an Order of the Court which directs it to do something no one knows how to do! New legislation is long overdue and is urgently required; new standards of purification should have regard mainly to the value of the watercourse as a source of public water supply, and to a less degree to fishing and general amenity.

Lack of hydrological data of our hard-worked, small, but all the more precious rivers, is a reproach to us, and the public health engineer is not alone in his constant need of information which does not exist. Unless new standards of purification are to be postponed indefinitely they will have to be set up and applied without knowledge of one of the most fundamental factors affecting them.

Now I have finished. My endeavour this evening has been to focus attention on some of the problems which confront us as civil engineers. Some concern us all, some concern only the few, but all are matters which merit thought and perhaps change, if we are to use our association together to the best advantage of our own future and the future of the world we serve. Many new problems will present themselves for solution, and it may even be that the nuclear age, in the dawn of which we seem to stand, will multiply them. Accumulated learning and experience aid us individually to meet and surmount our difficulties, but collectively we must look to professional fraternity to unite us in better service.

References

1. Trans. Instn Civ. Engrs, vol. 1 (1836), p. 21.
2. Min. Proc. Instn Civ. Engrs, vol. 4 (1845), p. 24.
3. Min. Proc. Instn Civ. Engrs, vol. 1 (1837–41), p. 7 (Session 1838).
4. *Ibid.*, p. 63 (Session 1839).
5. Min. Proc. Instn Civ. Engrs, vol. 12 (1852–53), p. 25.
6. Frederick Braithwaite, "On the Rise and Fall of the River Wandle; its Springs, Tributaries, and Pollution." Min. Proc. Instn Civ. Engrs, vol. 20 (1860–61), p. 191. *Discussion by* S. C. Homersham, p. 235.
7. See reference 5, p. 34.
8. James Green, "Account of the recent Improvements in the Drainage and Sewerage of Bristol." Min. Proc. Instn Civ. Engrs, vol. 7 (1848), p. 76.
9. *Ibid.*, p. 86.
10. See reference 5, p. 89.
11. J. W. Bazalgette, "On the Main Drainage of London, and the Interception of the Sewage from the River Thames." Min. Proc. Instn Civ. Engrs, vol. 24 (1864–65), p. 280. *Discussion by* E. Chadwick, p. 331.

Mr H. F. Cronin, Past-President, moved

"That the best thanks of the Institution be accorded to the President for his Address and that he be asked to permit it to be printed in the Proceedings of the Institution."

Beginning his remarks by the words "Mr President," he said that it was a very great pleasure to address Mr Watson by that title and to propose what was the first of many votes of thanks which the President would receive during his year of office, a year which they hoped and knew would be a happy one for him and a successful year for the Institution. No doubt the President would accept the first part of the motion, but he might say that in no circumstances would he allow his Address to be printed. Happily that was unthinkable. It would be a great misfortune for the Institution, for seldom if ever had the members listened to such an interesting and informative Address. The President, with his lucid style, had covered many subjects—the Institution, its foundation, its aims and objects, the obligations of the members to it, the reticence of engineers, that much-abused word "administration," contracts and contractors, and then Mr Watson's own realm, the realm of public health. Interspersed among these many subjects were a number of queries, which would give members furiously to think when they read the Address.

The President had dealt with the disposal of trade wastes, the pollution of rivers, sewage purification, and the application of sewage as a fertilizer to the land. We in this country knew very well that we were living in an over-populated island, and the problem of the use of sewage as a fertilizer was a very real one and one which, Mr Cronin felt sure, would engage the best brains of the Institution in the near future. According to the Board of Trade returns, this country used about 2,800,000 tons of artificial fertilizers each year, of which 700,000 tons had to be imported, at a cost of about £9,000,000.

The President had referred very modestly in his Address to the pride which he felt in following in the footsteps of his distinguished father, whose portrait looked down on the meeting from a wall of the lecture theatre. There had in the annals of the Institution been three other instances where son had followed father. There had been the two Hawksleys and the two Binnies, and it was interesting to record that those four gentlemen had all been concerned with public health. There had also been the two Hawkshaws, who had been railway and dock engineers respectively. On the marble slab on which Mr Watson's name would be recorded on the following morning, the name of Stephenson appeared twice, but they were not father and son but uncle and nephew.

Mr Cronin said in conclusion that the President had given a bonny Address, and that they were all very pleased to see Mrs Watson present that evening. Some of them knew that her health was not all that they would like it to be. They would all wish her a very happy year of office as "Mrs President" and hoped that she would grace their functions at the Institution and elsewhere with the President on many occasions.

Mr Ralph Freeman said that it was an honourable duty as well as his pleasure to second the motion which Mr Cronin had so ably proposed. No one who had listened to the Address, or who would read it later, could fail

to recognize it as a most notable contribution to the annals of the Institution, a contribution of no less merit than was to be expected from so distinguished an Engineer as Mr Watson, and one which upheld the high standard which Members had come to expect of the Presidential Addresses.

For the benefit of those who did not know all about the new President already, Mr Freeman had done a little research and had collected some facts about Mr Watson's career which might be of interest. He first saw the light of day in Aberdeen, but descended (in a geographical sense) to Birmingham to be educated and take his degree. His practical training found him first in Leeds and later, by way of a change, in New York. He had practised as a Consulting Engineer in Public Health since 1919. He was an acknowledged expert on sewerage and sewage disposal, as those who had heard his Address would realize, and had engineered schemes of all shapes and sizes for all sorts of Authorities scattered not only all over this country but far and wide in the Commonwealth and elsewhere abroad.

The preparation of the Inaugural Address was but a forerunner of the heavy labour and responsibility lying ahead of each new President at the outset of the term of office in the service of the Institution. On behalf of all present, Mr Freeman congratulated Mr Watson most heartily on his Address, and wished him all success and good fortune in the direction of the Institution's affairs.

The motion was carried with acclamation.

The President acknowledged the motion and gave permission for his Address to be printed in the Proceedings.

MEDALS AND PREMIUMS, SESSION 1953–54

For Papers presented for discussion at Ordinary Meetings :—

(1) A Telford Premium to Major-General G. N. Tuck, C.B., O.B.E., for his Paper on "The Engineer Task in Future Wars."
(2) A Telford Premium to Bryan Donkin, B.A., M.I.E.E., A. E. Margolis, Dipl.Ing., and C. G. Carrothers, B.Eng., M.I.Mech.E., M.I.E.E., jointly, for their Paper on "The Pimlico District Heating Undertaking."
(3) A Telford Premium to D. P. Bertlin, M.Eng., M.I.C.E., and Henry Olivier, C.M.G., M.Sc.(Eng.), Ph.D., M.I.C.E., jointly, for their Paper on "Owens Falls: Constructional Problems."
(4) A Crampton Prize to P. A. Scott, B.Sc., M.I.C.E., and J. I. Campbell, M.I.C.E., jointly, for their Paper on "Woodhead New Tunnel: Construction of a Three-Mile Main Double-Line Railway Tunnel."
(5) A Telford Premium to Sir Charles Westlake, M.I.E.E., and T. A. L. Paton, B.Sc.(Eng.), M.I.C.E., for the joint Paper by Mr R. W. Mountain and themselves on "Owen Falls, Uganda, Hydro-Electric Development" (special thanks of the Institution being accorded to Mr Mountain, Member of Council).

For Papers presented at Meetings of the Engineering Divisions :—

HYDRAULICS ENGINEERING DIVISION

(1) A Telford Premium to H. E. Hurst, C.M.G., M.A., D.Sc., for his Paper on "Measurement and Utilization of the Water Resources of the Nile Basin."
(2) A Trevithick Premium to E. A. G. Johnson, C.B.E., B.Sc.(Eng.), M.I.C.E., for his Paper on "Land Drainage in England and Wales."
(3) A Telford Medal posthumously to W. H. Godrey for the joint Paper by Mr C. A. Risbridger and himself on "Rainfall, Run-off, and Storage: Elan and Claerwen Gathering Grounds" (special thanks of the Institution being accorded to Mr Risbridger, Member of Council).

MARITIME AND WATERWAYS ENGINEERING DIVISION

(4) A Telford Premium to J. T. Williams, B.Sc.(Eng.), A.M.I.C.E., for his Paper on "Overhaul and Repair of Lock Gates in the Port of London."

PUBLIC HEALTH ENGINEERING DIVISION

(5) A Telford Premium to R. le G. Hetherington, O.B.E., M.A., M.I.C.E., and J. C. A. Roseveare, Jun., D.S.O., B.Sc.(Eng.), M.I.C.E., jointly, for their Paper on "The River Severn Scheme for the Water Supply of Coventry."

RAILWAY ENGINEERING DIVISION

(6) A Webb Prize to I. M. Campbell, B.Sc.(Eng.), A.M.I.C.E., and N. J. Nicholls, B.Sc.(Eng.), A.M.I.C.E., jointly, for their Paper on "Railway Civil Engineering in the United States of America."

ROAD ENGINEERING DIVISION

(7) The Coopers Hill War Memorial Prize to J. A. Neill, B.Sc.(Eng.), A.M.I.C.E., for the joint Paper by Mr Ralph Freeman and himself on "The Design and Construction of a High-Speed Test Track for Motor Vehicles." (Special thanks of the Institution being accorded to Mr Freeman, Member of Council.)

STRUCTURAL AND BUILDING ENGINEERING DIVISION

(8) A Telford Premium to T. J. Upstone, M.Sc., A.M.I.C.E., and W. N. Cardno, jointly, for their Paper on "The Design and Construction of the Superstructure of the Marshal Carmona Bridge at Vila Franca de Xira, Portugal."

WORKS CONSTRUCTION DIVISION

(9) A Manby Premium to O. A. Kerensky, B.Sc.(Eng.), M.I.C.E., and K. E. Hyatt, B.Sc.(Eng.), M.I.C.E., jointly for their Paper on "Design and Construction of Rama VI, Surat, and Bandara Bridges in Thailand."

For Papers published with and without written discussion in the Proceedings for 1953 :—

(1) A Telford Premium to R. E. Gibson, B.Sc.(Eng.), A.M.I.C.E., and Peter Lumb, M.Sc.(Eng.), Grad.I.C.E., jointly, for their Paper on "Numerical Solution of Some Problems in the Consolidation of Clay."
(2) A Telford Premium to H. D. Morgan, M.Sc.(Eng.), M.I.C.E., and C. K. Haswell, B.Sc.(Eng.), M.I.C.E., jointly, for their Paper on "The Driving and Testing of Piles."
(3) A Telford Premium to D. B. Waters B.Sc., A.M.I.C.E., for his Paper on "The Performance of Some Asphalt and Coated-Macadam Mixing Plant."
(4) A Telford Premium to T. P. O'Sullivan, Ph.D., B.Sc.(Eng.), M.I.C.E., for his Paper on "Strengthening of Steel Structures under Load."
(5) A Trevithick Premium to Professor A. L. L. Baker, B.Sc.Tech., M.I.C.E., for his Paper on "Further Research in Reinforced Concrete, and its Application to Ultimate Load Design."
(6) A Trevithick Premium to Jack Duvivier, B.Sc.(Eng.), M.I.C.E., for his Paper on "Coast Protection : Some Recent Works on the East Coast, 1942–52."

Graduates' and Students' Papers

For Papers read before Local Associations and the Association of London Graduates and Students :—

(1) The James Forrest Medal, the James Prescott Joule Medal, and a Miller Prize to D. H. Kent, Stud.I.C.E., for his Paper on "Models of Hydraulic Structures" (Association of London Graduates and Students).
(2) A Miller Prize to B. N. Harvey, Grad.I.C.E., for his Paper on "Concrete Work at Upper Glendevon Reservoir" (Edinburgh and East of Scotland Association).
(3) A Miller Prize to R. W. Rennison, B.Sc., Grad.I.C.E., for his Paper on "The Design and Construction of Rapid Gravity Filters" (Northern Counties Association).
(4) A Miller Prize to R. W. Drummond, B.Sc., Stud.I.C.E., for his Paper on "The Use of a Climbing Shutter on Luichart Surge Shaft" (Edinburgh and East of Scotland Association).
(5) A Miller Prize to B. A. M. Watt, Stud.I.C.E., for his Paper on "Construction of Retaining Wall" (Glasgow and West of Scotland Association).
(6) A Miller Prize to N. S. Lindsay, Grad.I.C.E., for his Paper on "A Report on a Levelling Operation" (Glasgow and West of Scotland Association).
(7) A Miller Prize to J. D. Addly, B.Sc., Grad.I.C.E., for his Paper on "Construction of Water Supply Works by Direct Labour in Dumfriesshire" (Edinburgh and East of Scotland Association).
(8) A Miller Prize to G. E. Peart, B.Sc.(Eng.), Grad.I.C.E., for his Paper on "Modern Methods of Permanent Way Maintenance and Renewal" (South-Western Association).
(9) A Miller Prize to William Sommerville, Stud.I.C.E., for his Paper on "Stream Intakes" (Glasgow and West of Scotland Association).

The Institution Medal and Premium (London University)

Awarded on the result of an annual competition between undergraduates of the University of London.

To Jeremy Christopher Joel, Stud.I.C.E., of King's College, who read a Paper entitled "Tunnelling under Compressed Air in Non-Cohesive Soils applied to a Large-Diameter Sewer."

The Institution Medal and Premium (Local Associations)

Awarded on the result of an annual competition between Graduates and Students of the Local Associations.

To Brian Norman Harvey, Grad.I.C.E., of the Edinburgh and East of Scotland Association, who read a Paper entitled "Concrete Work at Upper Glendevon Reservoir."

Paper No. 5989

"A Theoretical and Experimental Analysis of Sheet-Pile Walls"

by

*Peter Walter Rowe, Ph.D., A.M.I.C.E.

(Ordered by the Council to be published with written discussion)

SYNOPSIS

The variation of the maximum bending moment on sheet-pile walls with pile flexibility and soil stiffness is calculated for the cases of cantilevered piling and anchored piling, assuming a modulus of subgrade reaction which increases linearly with depth. The mathematics is too involved for direct design office use, but a final "master" moment/flexibility curve is calculated which is of universal application.

The value of the modulus of the soil is estimated from theory and simple stiff-wall tests, and results in good agreement between the theory and observations on model sheet-pile walls. The influence of seepage forces and more compressible subsoils than loose sand on the stability of sheet-piling is then readily estimated from observations of their influence on the soil modulus.

INTRODUCTION

ATTEMPTS to apply rigorous mathematical treatment to the problems of the deflexion of a flexible retaining wall freely embedded in a uniform bed of clean cohesionless material generally lead to equations so involved as to be unsuitable for direct practical application. In addition, soil in nature is usually far from being in a uniform, clean, or completely cohesionless state, so that very approximate methods of analysis have been considered to be of equal value.

It has been shown recently [1] that the bending moments, and anchor loads acting on an anchored sheet-pile wall are governed principally by the flexibility of the sheeting and the density of the subsoil for a given depth of penetration and soil to be retained. These factors may be taken into account in the design by the use of empirical reduction curves.

Whilst this method of design is an improvement for the particular case of anchored walls driven in sands and gravels, there is need for a theoretical treatment of flexible wall structures which can be applied to both anchored and cantilevered walls, extended to braced and filled cofferdams, and used to predict the influence of seepage forces and of silt in the subsoil.

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[1] The references are given on p. 67.

Such a theory must take pile flexibility and soil stiffness into account, but should not lead to a final form so involved as to be unusable.

The fundamental equation governing the flexure of a beam supported by soil is :

$$EI\frac{d^4y}{dx^4} = p_a - p_b = f(xy) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

To solve the equation, it is necessary first to determine the nature of the passive pressure distribution p_b in terms of x and y. Experimental and theoretical investigations [2] have shown that p_b varies non-linearly with both x and y and substitution of the calculated function in x and y, or of many proposed equivalent empirical formulae into equation (1) leads to either a solution of outstanding complexity, or no solution at all. The simple empirical formula :

$$p_b = Kxy^n \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

where K and n are constants, does, however, form a fairly accurate substitute for the observed and theoretical variation in pressure with x and y for cohesionless soil. Rifaat [3] solves equation (1) using equation (2a) for the case of the cantilever pile and Blum [4] solves the case of the anchored pile taking $n = 1$. Both solutions are too complicated for general use and the values of the soil constant are unknown.

The quantity K in equation (2a) cannot be constant at all model scales, since the pressure p_b is known to vary approximately linearly with scale (or even to a lower power than unity with scale) and not with the square of the scale as the equation implies ($n = 1$). The increase in p_b with depth x reflects the increase of shear strength with confining pressure, an increase which is not quite linear.[2] The increase with deflexion y reflects the increase in mobilization of ϕ with shear slip strain which is a function of the ratio of the movement y to the length of the slip path.[2] Thus if one wall is identical with another but twice the size, then the deflexions y at corresponding depths below the dredge level are in the ratio 2 to 1, but so are the lengths of the slip paths, so that the pressure ratio is uninfluenced by the deflexions. Since the slip path length for a given point (x, y) is related linearly to the scale or depth of penetration D, equation (2a) may be rewritten :

$$p_b = \frac{mxy^n}{D^n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2b)$$

The value of n is less than unity, but quite good agreement is obtained with observations on models [2, 3] and with theoretical pressure distributions based on shear-box and triaxial compression tests, taking $n = 1$. This is due to the fact that the difference between the equivalent straight line relation, taking $n = 1$ and the observed curve, for the range of movements likely in practice, is very small compared with the large variations in the value of the coefficient m with soil state. Since p_b does not vary

strictly linearly with x either, the use of equation (2b) is at best an approximation which is dictated by the necessity to solve equation (1). It is proposed to use $n = 1$ on the understanding that the value of m will decrease with large increases in x and y. The exact values and variations in the value of m are discussed after the calculations are completed.

Notation

$a_{0, 1, 2 \ldots n}$	denote	arbitrary coefficients.
D	denotes	penetration depth.
D_r	,,	relative soil density.
E	,,	Young's modulus.
$f_1 - f_5$	denote	functions of χ.
H	denotes	height of pile.
I	,,	section moment of area.
K_a	,,	active earth-pressure coefficient.
K_p	,,	passive earth-pressure coefficient.
K_{T1}, K_{T2}	denote	functions of χ and α due to point load.
K_{P1}, K_{P2}	,,	functions of χ and α due to active earth pressure.
K_{S1}, K_{S2}	,,	functions of χ and α due to surcharge pressure.
M	denotes	bending moment.
m	,,	soil stiffness modulus.
p_a	,,	resultant active pressure below the dredge level.
p_p	,,	resultant passive pressure below the dredge level.
q_s	,,	surcharge pressure.
q	,,	surcharge coefficient : $\frac{q_s}{\gamma H}$.
T	,,	tie-rod or point load.
x	,,	depth below dredge level.
y	,,	horizontal deflexion at point x.
y_A	,,	deflexion at anchor level.
y_a	,,	total deflexion at top of cantilever pile.
y_b	,,	component of y_a due to bending above the dredge level.
y_d	,,	deflexion at the dredge level.
y_S	,,	component of y_a due to slope at the dredge level.
Z	,,	depth below top of pile.
α	,,	$1 - \frac{D}{H}$.
β	,,	the ratio : $\frac{\text{depth to anchor-level}}{\text{pile height}}$.
γ	,,	earth density.
ϵ	,,	$\frac{x}{D}$.

λ	,,	$\dfrac{T}{H^2}$.
λ'	,,	$\dfrac{T}{K_a\gamma H^2}$.
μ	,,	Poisson's ratio.
η	,,	$\dfrac{Z}{H}$.
ρ	,,	flexibility number : $\dfrac{H^4}{EI}$.
τ	,,	$\dfrac{M}{H^3}$.
τ'	,,	$\dfrac{M}{K_a\gamma H^3}$.
τ_m, τ'_m	denote	maximum values of τ, τ'.
χ	denotes	$\dfrac{1}{10}(1-\alpha)^4 m\rho\epsilon^5$.
χ_1	,,	$\dfrac{1}{10}(1-\alpha)^4 m\rho$.
ϕ	,,	coefficient of friction.

Theoretical Solution

Equation (1) may be solved by relaxation, or by expanding the pressure p_p in an infinite series. By either of these methods the solution is simple and rapid for stiff piling, but increasing degrees of accuracy are required with increase in pile flexibility and soil stiffness. For flexible piling the relaxation method leads to the use of large operating numbers required to cancel very small residuals, and the arithmetical accuracy required is unsuitable for the trial and error process of relaxation. In addition, it is necessary to repeat the full analysis for each individual problem.

Blum [4] attempted to produce a solution simple enough for design purposes by writing: $p_p = Ax^3 + Bx^2 + Cx$. This is equivalent to writing: $y = A'x^2 + B'x + C'$. He demonstrated that, for a cantilevered pile of medium flexibility, no great difference occurred if the calculation were repeated taking an expansion one order higher, that is to say, $y = A'x^3 +$ etc., and concluded that for simplicity the second order expansion was sufficient.

However, comparison of the results of the second order expansion and a thirty-order expansion show larger differences at medium flexibilities, and the low order expansion leads to large errors with higher flexibilities

in the practical range. In addition, even the use of a second order expansion leads to calculations unsuitable for repetition in a design office.[5] For these reasons a thirty-order expansion has been used, and in order to maintain the calculations within satisfactory limits of arithmetical accuracy over the practical limits of flexibility and soil stiffness, the calculations were carried through to ten consecutive figures on a machine. The intermediate equations are generally in a form unsuitable for general use, but lead to the establishment of a few final graphs which are of universal application and simple to use. Thus, the calculation need never be repeated.

In view of the lengthy nature of the thirty-order expansion, only the first few terms of each series are reproduced once the form of the expansion is established.

Substituting p_p, equation (2*b*), in equation (1), taking $n = 1$, it is found that:

$$EI\frac{d^4y}{dx^4} = p_a - \frac{mxy}{(1-\alpha)H} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$\left.\begin{aligned}
&\text{Let} \quad y = a_0 + a_1x + a_2x^2 + a_3x^3 \ldots + a_nx^n \quad . \quad . \quad . \quad (a)\\
&\text{Then} \quad \frac{dy}{dx} = a_1 + 2a_2x + 3a_3x^2 + 4a_4x^3 \ldots + (n+1)a_{(n+1)}x^n \\
&\qquad . \quad . \quad . \quad . \quad . \quad (b)\\
&\frac{d^2y}{dx^2} = 2a_2 + 3!a_3x + \frac{4!}{2!}a_4x^2 + \frac{5!}{3!}a_5x^3 \ldots \\
&\qquad + \frac{(n+2)!}{n!}a_{(n+2)}x^n \quad . \quad . \quad . \quad (c)\\
&\frac{d^3y}{dx^3} = 3!a_3 + 4!a_4x + \frac{5!}{2!}a_5x^2 + \frac{6!}{3!}a_6x^3 \ldots \\
&\qquad + \frac{(n+3)!}{n!}a_{(n+3)}x^n \quad . \quad . \quad . \quad . \quad (d)\\
&\frac{d^4y}{dx^4} = 4!a_4 + 5!a_5x + \frac{6!}{2!}a_6x^2 + \frac{7!}{3!}a_7x^3 \ldots \\
&\qquad + \frac{(n+4)!}{n!}a_{(n+4)}x^n \quad . \quad . \quad . \quad . \quad (e)
\end{aligned}\right\} (4)$$

Substituting equation (4*e*) for $\frac{d^4y}{dx^4}$ in the left-hand side of equation (3) and equation (4*a*) for y in the right-hand side, gives:

$$\left[4!a_4 + 5!a_5x + \frac{6!}{2!}a_6x^2 \ldots\right] = \frac{p_a}{EI} - \frac{mx}{(1-\alpha)HEI}[a_0 + a_1x + a_2x^2 \ldots] \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

By equating coefficients in $x, x^2 \ldots x^n$, the values of all the coefficients $a_5, a_6 \ldots a_n$ are obtained in terms of the five coefficients $a_0 - a_4$, and equations (4) may be rewritten as a series in terms of these five coefficients.

For example, writing: $\mathrm{A} = \dfrac{m}{(1-\alpha)HEI}$

$$y = a_0\left[1 - \frac{\mathrm{A}}{5!}x^5 + \frac{6!}{10!5!}\mathrm{A}^2x^{10}\ldots\right]$$

$$+ a_1x\left[1 - \frac{2!}{6!}\mathrm{A}x^5 + \frac{7!2!}{11!6!}\mathrm{A}^2x^{10}\ldots\right]$$

$$+ a_2x^2\left[1 - \frac{3!}{7!}\mathrm{A}x^5 + \frac{8!3!}{12!7!}\mathrm{A}^2x^{10}\ldots\right]$$

$$+ a_3x^3\left[1 - \frac{4!}{8!}\mathrm{A}x^5 + \frac{9!4!}{13!8!}\mathrm{A}^2x^{10}\ldots\right]$$

$$+ a_4x^4\left[1 - \frac{5!}{9!}\mathrm{A}x^5 + \frac{10!5!}{14!9!}\mathrm{A}^2x^{10}\ldots\right] \quad \ldots\ldots (6)$$

Since the term $\left(\dfrac{\mathrm{A}x^5}{10}\right)$ occurs throughout this substitute may be used:

$$144\chi = \frac{\mathrm{A}x^5}{10} = \frac{m}{10(1-\alpha)HEI}\cdot(\epsilon(1-\alpha)H)^5 = \frac{1}{10}\cdot\frac{H^4}{EI}\cdot m(1-\alpha)^4\epsilon^5.$$

The coefficietss χ, α, and ϵ are dimensionless and the dimensions of m and $\left(\dfrac{EI}{H^4}\right)$ are $\dfrac{\text{lb.}}{\text{ft}^3}$. It is of practical convenience, however, to express EI in the units lb. inch²/ft and the height H in feet, and write:

$$\rho = \frac{H^4}{EI}\ \frac{\text{ft}^5}{\text{lb. inch}^2}.$$

Hence
$$\chi = \frac{\mathrm{A}x^5}{1440} = \frac{1}{10}m\rho(1-\alpha)^4\epsilon^5 \quad \ldots\ldots (7)$$

Equations (4) are solved therefore in terms of coefficients $a_0 - a_4$ and χ, which in turn are a function of the relative stiffness of the pile and the soil.

The coefficients a_2, a_3, and a_4 are obtained directly by inspection of the boundary conditions of the particular problem. In addition, for every problem considered, the bending moment and shear force on the pile at the toe are zero. Hence the remaining coefficients a_0 and a_1 are obtained in each case by writing:

$$\frac{d^2y}{dx^3} = 0 \quad \text{and} \quad \frac{d^3y}{dx^3} = 0, \quad \text{when } x = (1-\alpha)H$$

for equations (4c) and (4d). It is only necessary, therefore, to state the values of the coefficients $a_2 - a_4$ for each case and to give the final solutions.

Suitable combinations of the solutions to the following three main cases may be used to solve the cantilever pile, the anchored pile with any tie-rod level, surcharge, and anchor yield, and lead to a solution of the double-walled cofferdam.

Case 1.—A cantilever pile subject to a line load at the top.

Case 2.—A cantilever pile subject to triangular loading above the dredge level.

Case 3.—A cantilever pile subject to rectangular loading above the dredge level.

Case 1.—A cantilever pile subject to a line load at the top (*Fig. 1*)

Fig. 1

Boundary conditions at $x = 0$.

Bending moment $= T\alpha H = EI\dfrac{d^2y}{dx^2}$.

From equation (4c) : $$a_2 = \frac{T\alpha H}{2EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

Shear force $= T = EI\dfrac{d^3y}{dx^3}$.

From equation (4d) : $$a_3 = \frac{T}{6EI} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

The earth pressure at rest on either side of the piling is equal, so that, $p_a = 0$.

From equation (4e) : $$a_4 = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

The two solutions of interest are the distribution of bending moment below the dredge level and the outward deflexion of the top of the wall.

Writing $\tau = \frac{M}{H^3}$, where the units are M lb. feet/foot and H feet,

and $\lambda = \frac{T}{H^2}$ lb. per foot^3,

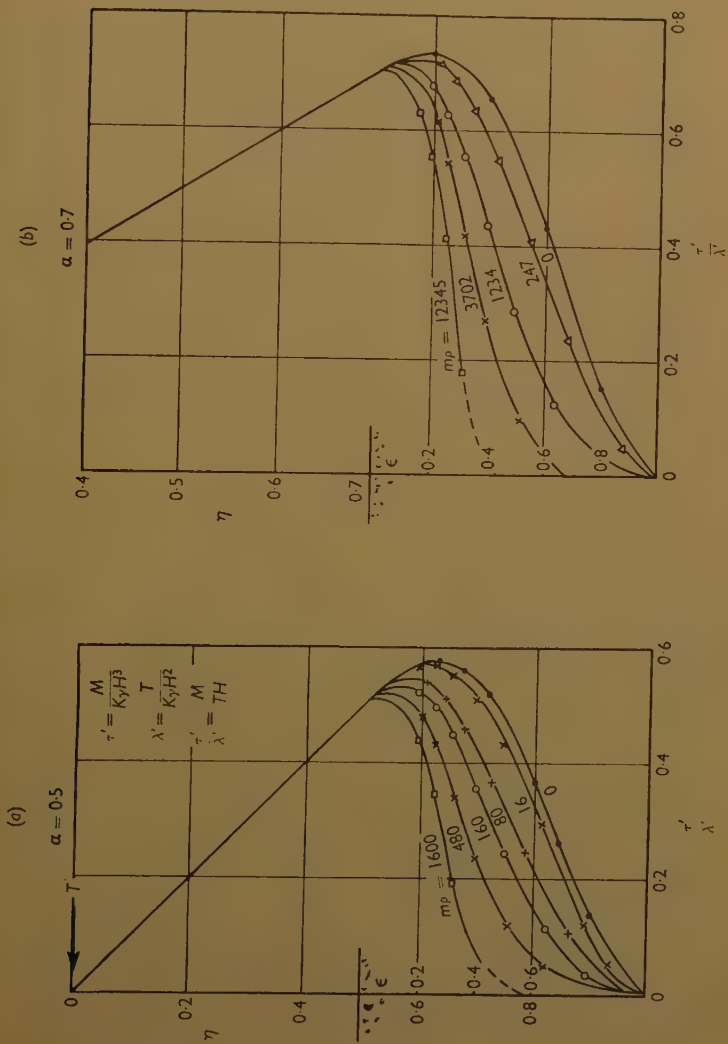


Fig. 2

CANTILEVER WITH POINT LOAD AT TOP

the solution of equation (4c) gives the following bending-moment distribution beneath the dredge level :

$$\frac{T}{\lambda} = \alpha f_1 + (1-\alpha)\epsilon f_2 + 0\epsilon^2 f_3 - K_{T_1}\epsilon^3 f_4 + K_{T_2}\epsilon^4 f_5 \quad . \quad . \quad (11)$$

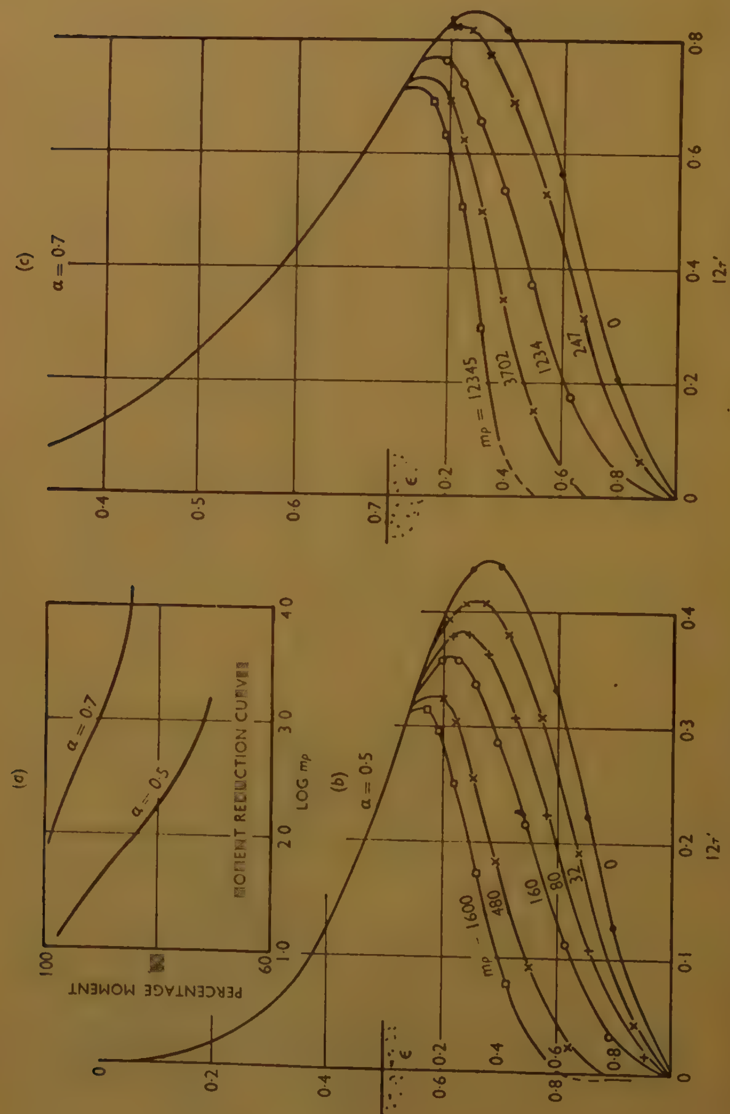


Fig. 3

CANTILEVER WITH TRIANGULAR ACTIVE PRESSURE

The values of $f_1 - f_5$ and K_{T_1} and K_{T_2} are given in the Appendix and in *Figs 24–27*. The bending-moment distributions are plotted in *Fig. 2* for the cases $\alpha = 0{\cdot}5$ and $\alpha = 0{\cdot}7$ for a wide range of values of $m\rho$.

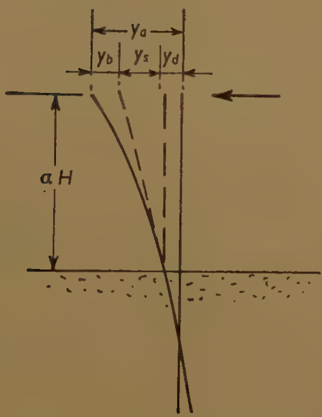
The deflexion at the point of action of the load at the top of the pile (*Fig. 4*) is given by:

$$y_a = y_d + y_b + y_s \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

The deflexion at the dredge level y_d is given by equation (4*a*):

$$y_d = a_0 = \frac{T}{H^2}\,\frac{6H}{m(1-\alpha)^2}K_{T_1} \quad . \quad . \quad . \quad . \quad (13)$$

Fig. 4



The slope at the dredge level is given by equation (4*b*):

$$\left(\frac{dy}{dx}\right)_{x=0} = a_1 = -\frac{T}{H^2}\,\frac{12}{m(1-\alpha)^3}K_{T_2} \quad . \quad . \quad . \quad (14)$$

The deflexion at the top due to slope at the dredge level is therefore:

$$y_s = -\alpha H\left(\frac{dy}{dx}\right)_{x=0} = +\frac{T}{H^2}\,\frac{12\alpha H}{m(1-\alpha)^3}K_{T_2} \quad . \quad . \quad . \quad (15)$$

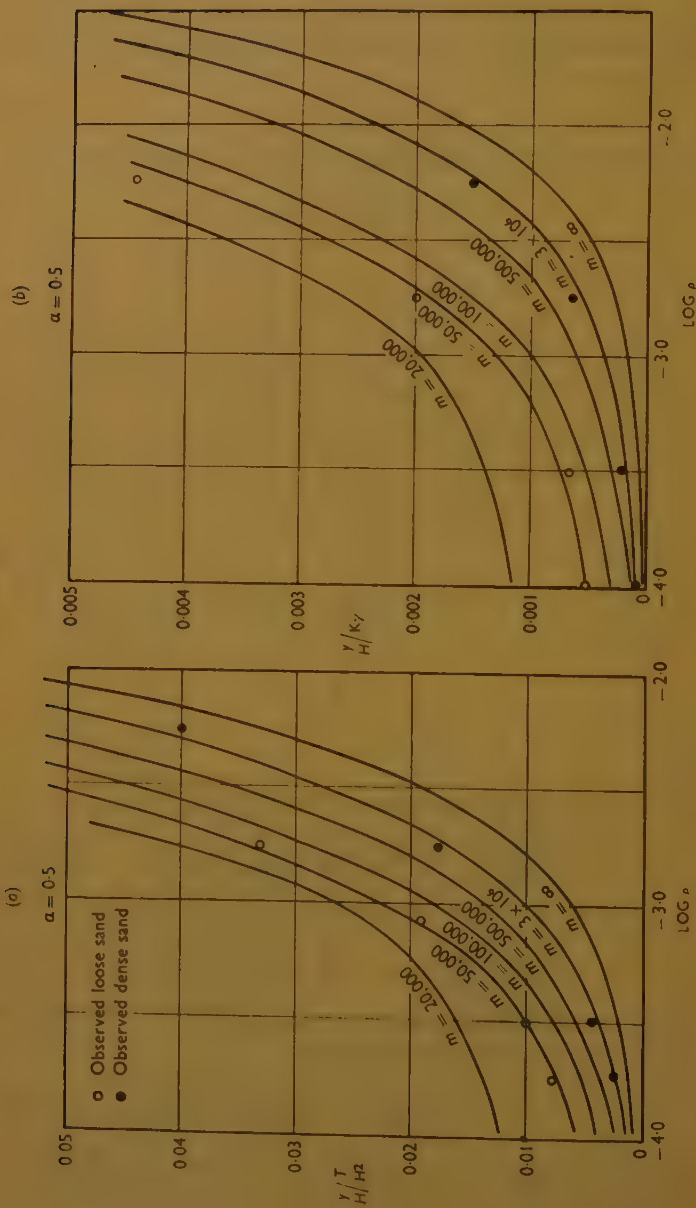
The bending deflexion between the dredge level and the top of the wall is obtained from simple bending theory:

$$y_b = \frac{T(\alpha H)^3}{3EI} = 48\alpha^3\rho\,\frac{T}{H^2}H \quad . \quad . \quad . \quad . \quad (16)$$

Substituting equations (13), (15), and (16) in equation (12) gives:

$$\left(\frac{y_a}{H}\right)_T = \frac{T}{H^2}\left[48\alpha^3\rho + \frac{6}{(1-\alpha)^2}\,\frac{K_{T_1}}{m} + \frac{12\alpha}{(1-\alpha)^3}\,\frac{K_{T_2}}{m}\right] \quad . \quad (17)$$

Fig. 5



UNIT DEFLEXIONS AT TOP OF CANTILEVER

(*a*) POINT LOAD: (*b*) TRIANGULAR ACTIVE PRESSURE

Values of $(y_a/H)_T$ divided by λ are plotted in *Fig. 5* for various values of $\log \rho$ and m.

It is important to consider the degree of arithmetical accuracy required to use equations (11) and (17). For very stiff piles, that is to say, low values of χ_1, or $m\rho$, the second, third and fourth terms in equation (11) are small compared to the first, and slide-rule accuracy is sufficient. With higher values of χ_1 the succeeding terms increase in size and become large towards the toe of the pile. In this region the bending moment is obtained from the difference of large numbers, so that much higher accuracy is required. Fortunately it will rarely be found that the exact moment distribution near the toe is required. The maximum moment near the dredge level may be calculated as a ratio of the maximum value for a stiff pile and plotted as a theoretical reduction curve (see *Fig. 3* (a)). The solution of equation (17) may be obtained satisfactorily by slide-rule accuracy using values of K_{T_1} and K_{T_2} interpolated from the graphs in the Appendix. It is necessary to use values of K_{T_1} and K_{T_2} to ten consecutive figures when equation (11) is being solved to the pile toe for flexible piling.

Case 2.—A cantilever pile subject to a triangular pressure distribution (*Fig. 6*)

Proceeding as for *Case 1*:

Fig. 6

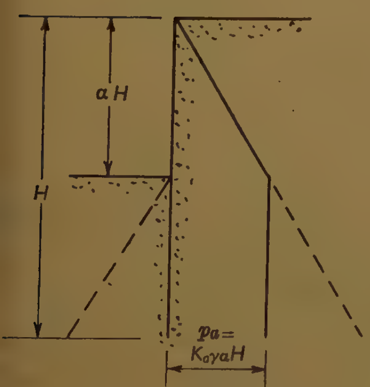
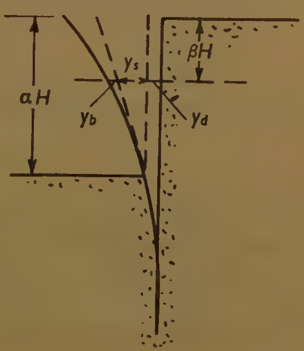


Fig. 7



Boundary conditions at $x = 0$.

$$\text{Bending moment} = \frac{K_a\gamma(\alpha H)^3}{6} = EI\frac{d^2y}{dx^2}$$

$$a_2 = \frac{1}{12}\frac{K_a\gamma}{EI}(\alpha H)^3.$$

$$\text{Shear force} = \frac{K_a\gamma(\alpha H)^2}{2} = EI\frac{d^3y}{dx^3}$$

$$a_3 = \frac{1}{12}\frac{K_a\gamma}{EI}(\alpha H)^2.$$

Resultant active earth pressure below the dredge level (*Fig. 6*) is constant and given by:

$$p_a = K_a\gamma\alpha H = EI\frac{d^4y}{dx^4},$$

$$a_4 = \frac{1}{4!}\frac{K_a\gamma\alpha H}{EI}.$$

Substituting these values in equations (6) and solving for a_0 and a_1, the distribution of bending moment beneath the dredge level is given by:

$$\tau'\left(\frac{6}{\alpha}\right) = \alpha^2 f_1 + 3\alpha(1-\alpha)\epsilon f_2 + 3(1-\alpha)^2\epsilon^2 f_3 - K_{P_1}\epsilon^3 f_4 + K_{P_2}\epsilon^4 f_5 \quad (18)$$

The values of the coefficients K_{P_1} and K_{P_2} are given in the Appendix and in *Figs 24–27*. The distributions are plotted in *Fig. 3*. In order that the results may be subsequently extended to the anchored pile, it is necessary to determine the outward deflexion of the cantilever, y_β, at a point distant βH below the top of the pile (see *Fig. 7*).

The component of y_β due to deflexion at the dredge level is:

$$y_d = a_0 = \frac{K_a\gamma\alpha H}{m(1-\alpha)^2}K_{P_1} \quad (19)$$

The component due to slope at the dredge level is:

$$y_s = -(\alpha-\beta)Ha_1 = \frac{2\alpha(\alpha-\beta)K_a\gamma}{m(1-\alpha)^3}K_{P_2}\,.\,H \quad (20)$$

The component due to bending between points βH and αH from simple bending under the trapezoidal pressure distribution is:

$$y_b = 1{\cdot}2K_a\gamma\rho[\beta^5 - 5\alpha^4\beta + 4\alpha^5] \quad (21)$$

Substituting equations (19) to (21) in equation (12) gives:

$$\left[\frac{y_\beta}{H}\right]_P = K_a\gamma\left\{1{\cdot}2\rho[\beta^5 - 5\alpha^4\beta + 4\alpha^5] + \frac{\alpha}{m(1-\alpha)^2}K_{P1} + \frac{2\alpha(\alpha-\beta)}{m(1-\alpha)^3}K_{P_2}\right\} \quad (22)$$

Values of $\frac{y_\beta}{H}$ divided by $K_a\gamma$ are plotted in *Fig. 5* for values of $\log\rho$ and m.

Case 3.—A cantilever pile subject to rectangular loading above the dredge level

Boundary conditions at $x = 0$.

$$EI\frac{d^2y}{dx^2} = qK_a\gamma H\frac{(\alpha H)^2}{2}$$

$$a_2 = \frac{qK_a\gamma H^3}{EI}\frac{\alpha^2}{4}.$$

$$EI\frac{d^3y}{dx^3} = qK_a\gamma H^2\alpha$$

$$a_3 = q\frac{K_a\gamma H^2\alpha}{6EI}.$$

Considering the active pressure below the dredge level, no deduction should be made for the initial " at rest " pressure on the outside of the pile. The pressure diagram *Fig. 8* represents active pressure due to

Fig. 8

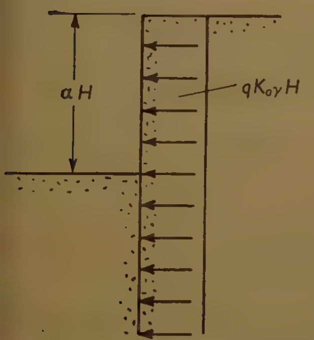
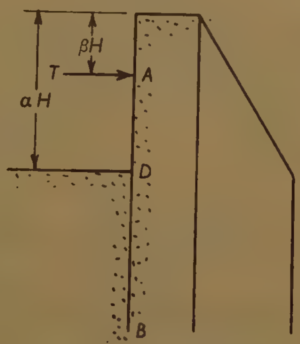


Fig. 9



surcharge of superimposed loads, or differential water pressure, and is additional to either *Case 1* or *Case 2*. In both these first two cases the triangular type, " at rest " pressure diagrams, balance out below the dredge level. The additional active pressure p_a is therefore constant,

and $EI\dfrac{d^4y}{dx^4} = qK_a\gamma H$; $a_4 = \dfrac{qK_a\gamma H}{4!EI}$.

The increment of bending-moment distribution is given by :

$$\frac{\tau'}{q} = \frac{\alpha^2}{2}f_1 + \alpha(1-\alpha)\epsilon f_2 + \tfrac{1}{2}(1-\alpha)^2\epsilon^2 f_3 - K_{S_1}\epsilon^3 f_4 + K_{S_2}\epsilon_4 f_5 \quad (23)$$

The values of the coefficients K_{S_1} and K_{S_2} are given in the Appendix and in *Figs 24–27.*

The components of deflexion at point βH below the top of the wall are:

$$y_d = a_0 = \frac{6qK_a\gamma HK_{S_1}}{m(1-\alpha)^2}$$

$$y_s = -(\alpha-\beta)Ha_1 = +\frac{12qK_a\gamma K_{S_2}}{m(1-\alpha)^3}(\alpha-\beta)H$$

$$y_b = 6qK_a\gamma H\rho(\beta^4 - 4\alpha^3\beta + 3\alpha^4).$$

The total outward deflexion at depth βH is given by:

$$\left[\frac{y_\beta}{H}\right]_S = qK_a\gamma\left\{6\rho(\beta^4 - 4\alpha^3\beta + 3\alpha^4) + \frac{6K_{S_1}}{m(1-\alpha)^2} + \frac{12(\alpha-\beta)}{m(1-\alpha)^3}K_{S_2}\right\} \quad (24)$$

Direct combinations of the solutions to *Cases 1* to *3* are sufficient to determine the stresses and deflexions of cantilever piling.

Case 4.—The anchored sheet-pile wall

The solution is obtained by resolving the forces on the wall into the three cases solved for cantilever piling and by equating the resultant deflexions at the anchor level.

Let Y_A = natural outward deflexion of the anchorage.

Then
$$\frac{Y_A}{H} = \left[\frac{y_\beta}{H}\right]_P + \left[\frac{y_\beta}{H}\right]_S - \left[\frac{y_\beta}{H}\right]_T \quad (25)$$

The values of the components $\left[\frac{y_\beta}{H}\right]_P$ and $\left[\frac{y_\beta}{H}\right]_S$ are given by equations (22) and (24) respectively. The value of $\left[\frac{y_a}{H}\right]_T$ given by equation (17), however, gives the value of the deflexion at the top of a pile where the point load is acting. The height of this pile is equivalent to AB (*Fig. 9*), so that the length $(1-\beta)H$ must be substituted for H in equation (17). Similarly the dredge-level coefficient is given by:

$$\frac{\mathrm{AD}}{\mathrm{AB}} = \frac{(\alpha-\beta)H}{(1-\beta)H} = \frac{\alpha-\beta}{1-\beta}$$

and this value must be substituted for α.

Equation (13) becomes:

$$y_d = \frac{T}{(1-\beta)^2H^2}\,\frac{6(1-\beta)H}{m\left[\left(1-\frac{(\alpha-\beta)}{(1-\beta)}\right]^2\right.}K_{T_1} = \frac{T}{H^2}\,\frac{6H}{m(1-\alpha)^2}(1-\beta)K_{T_1} \quad (26)$$

Equation (15) becomes:

$$y_s = \frac{T}{(1-\beta)^2 H^2} \frac{12(\alpha-\beta)}{(1-\beta)} \frac{(1-\beta)H}{\left[1-\frac{(\alpha-\beta)}{(1-\beta)}\right]^3} K_{T_2}$$

$$= \frac{T}{H^2} \frac{12}{m} \frac{(1-\beta)(\alpha-\beta)}{(1-\alpha)^3} K_{T_2} . H \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

Equation (16) becomes:

$$y_b = \frac{T}{3EI}\left[\frac{(\alpha-\beta)}{(1-\beta)} . (1-\beta)H\right]^3 = \frac{T}{H^2} 48\rho(\alpha-\beta)^3 . H. \quad (28)$$

The deflexion at the point of action of the load T at depth βH is given by:

$$\left[\frac{y\beta}{H}\right]_T = \frac{y_b + y_s + y_d}{H}$$

$$= \frac{T}{H^2}\left\{48\rho(\alpha-\beta)^3 + \frac{6(1-\beta)}{m(1-\alpha)^2} K_{T_1} + \frac{12(1-\beta)(\alpha-\beta)}{m(1-\alpha)^3} K_{T_2}\right\} \quad (29)$$

Substituting equations (22), (24), and (29) in equation (25), gives the final expression for the tie-rod load for any condition of anchor yield, anchor depth, surcharge, dredge level, pile flexibility, subsoil stiffness, and active earth pressure:

$$\lambda' = \frac{T}{K_a \gamma H^2} = \frac{\left[m\rho\{1{\cdot}2\beta^4(\beta+5q) - 6\alpha^3\beta(\alpha+4q) + 4{\cdot}8\alpha^4(\alpha+3{\cdot}75q)\} + \frac{1}{(1-\alpha)^2}[\alpha K_{P_1} + 6qK_{S_1}] + \frac{2(\alpha-\beta)}{(1-\alpha)^3}[\alpha K_{P_2} + 6qK_{S_2}] - \frac{Y_A}{H} . \frac{m}{K_a\gamma}\right]}{48m\rho(\alpha-\beta)^3 + \frac{6(1-\beta)}{(1-\alpha)^2} K_{T_1} + \frac{12(\alpha-\beta)(1-\beta)}{(1-\alpha)^3} K_{T_2}}$$

$$. \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

The maximum positive bending moment will always occur above the dredge level. Taking moments between the anchorage and the dredge level due to the active pressures and the tie-rod load, the maximum unit bending moment is given by:

$$\tau' = \frac{M_{max}}{K_a \gamma H^3} = \lambda'(\bar{\eta} - \beta) - \frac{\bar{\eta}^3}{6} - q\frac{\bar{\eta}^2}{2} \quad . \quad . \quad (31)$$

where

$$\bar{\eta} = \sqrt{(q^2 + 2\lambda')} - q \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

Equations (32) and (31) are readily solved once the value of λ' has been obtained from equation (30). The latter equation may be solved to three-figure accuracy using interpolated values of the coefficients K_T,

K_P, and K_S from *Figs 24–27* (see Appendix). The solution depends upon the anchor yield $\frac{Y_A}{H}$, which may vary in practice between values of 0 and 0·004, depending on the anchorage conditions. It is convenient to calculate the influence of the soil stiffness and pile flexibility on the bending moments and tie-rod loads for the case of no anchor yield and then to study the influence of yield on these results.

Equations (30), (31), and (32) have been solved for the conditions $q = 0$, $\beta = 0$, 0·1, and 0·2; and $q = 0{\cdot}2$, $\beta = 0{\cdot}1$, 0·2; for values of χ_1 equal to 0, 0·02, 0·05, 0·10, 0·20, 0·50, 1·00, 3·00, and 10·00 for $\frac{Y_A}{H} = 0$, 0·003, and 0·008. All these cases have been repeated for $\alpha = 0{\cdot}6$, 0·7, and 0·8. The case of $\chi_1 = 0$ represents an infinitely stiff pile. For $\frac{Y_A}{H} = 0$, this gives bending-moment and tie-rod load values about 2 per cent greater than those calculated assuming free earth support. This is because, with no anchor yield, the stiff wall rotates about the anchor with maximum deflexion at the toe. The passive pressure distribution then has a resultant slightly lower than the two-thirds point. Yields of the order of $\frac{H}{1000}$ for $\alpha = 0{\cdot}7$ give maximum bending moments exactly equal to the free earth support value for loose sand, from the theory. Quite apart from this, the free earth support calculation provides a simple practical basis on which to commence the design of anchored sheet-piling. The value of λ' by free earth support has been shown [1] to be given by:

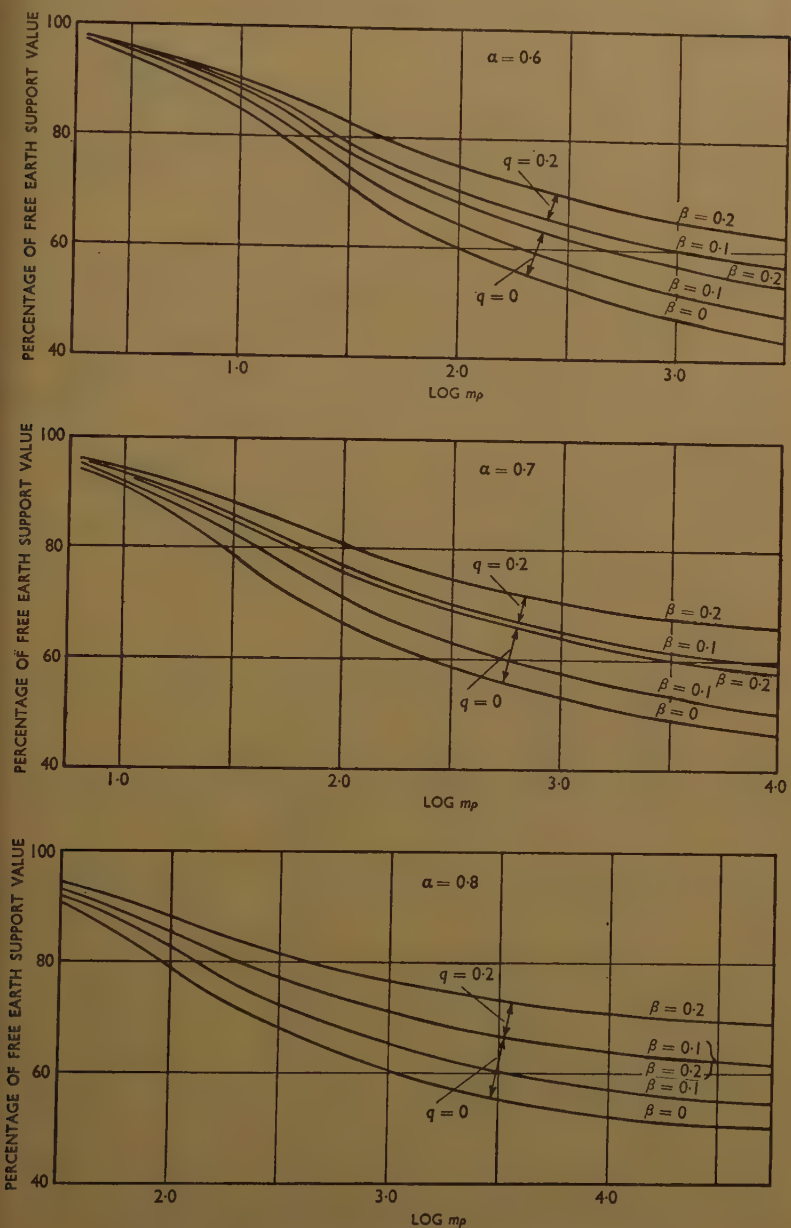
$$\lambda' = \tfrac{1}{2}\left[\frac{\alpha + q(1 + 2\alpha)}{2 + \alpha - 3\beta}\right] \qquad . \quad . \quad . \quad . \quad . \quad . \qquad (33)$$

and the corresponding maximum bending moment is obtained using equations (31) and (32). The values obtained of the actual λ' and τ' values on the structure have been divided by the corresponding free earth support values and the tie-rod loads and bending moments expressed as a percentage reduction on the free-earth-support values in *Figs 10* and *11*.

The theoretical bending-moment reduction values for the practical range of β and q values calculated, do not vary by more than $\pm$ 3 per cent on a mean reduction curve for a given value of α. In addition the general range of necessary penetration depths leads to values of α between 0·65 and 0·75, and in this range the theoretical reduction values are within $\pm$ 10 per cent of the mean reduction curve for $\alpha = 0{\cdot}7$.

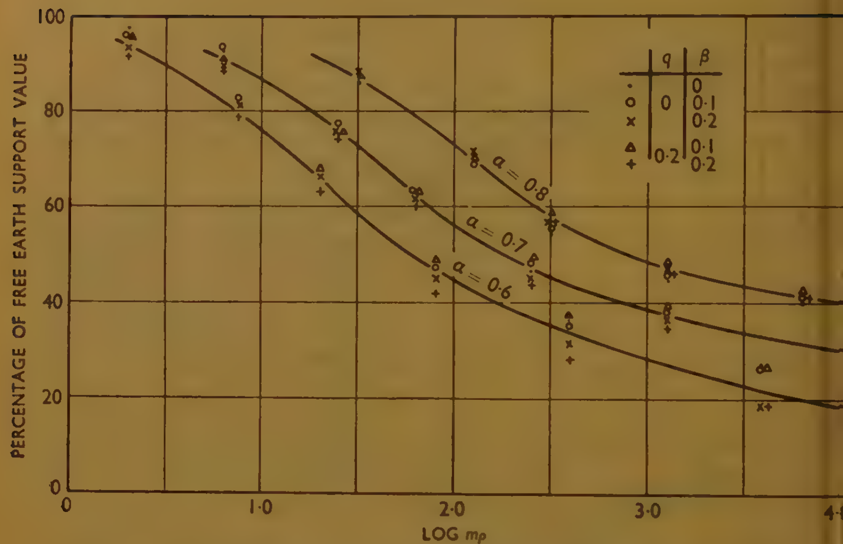
This theoretical result, which agrees with experimental observations,[1] means that one master reduction curve used in conjunction with the free earth support analysis may replace the full mathematical analysis

Fig. 10



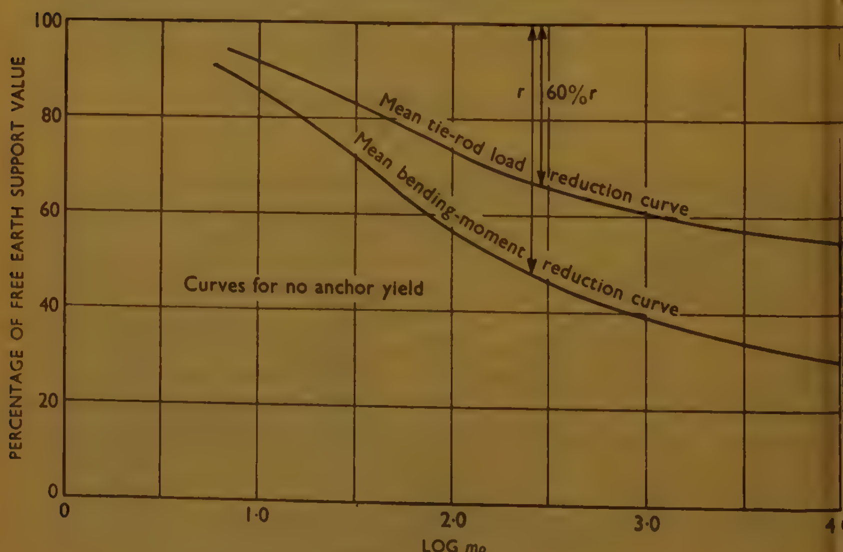
TIE-ROD LOAD VARIATION WITH FLEXIBILITY AND SOIL MODULUS

Fig. 11



Mean percentage bending-moment/flexibility curves

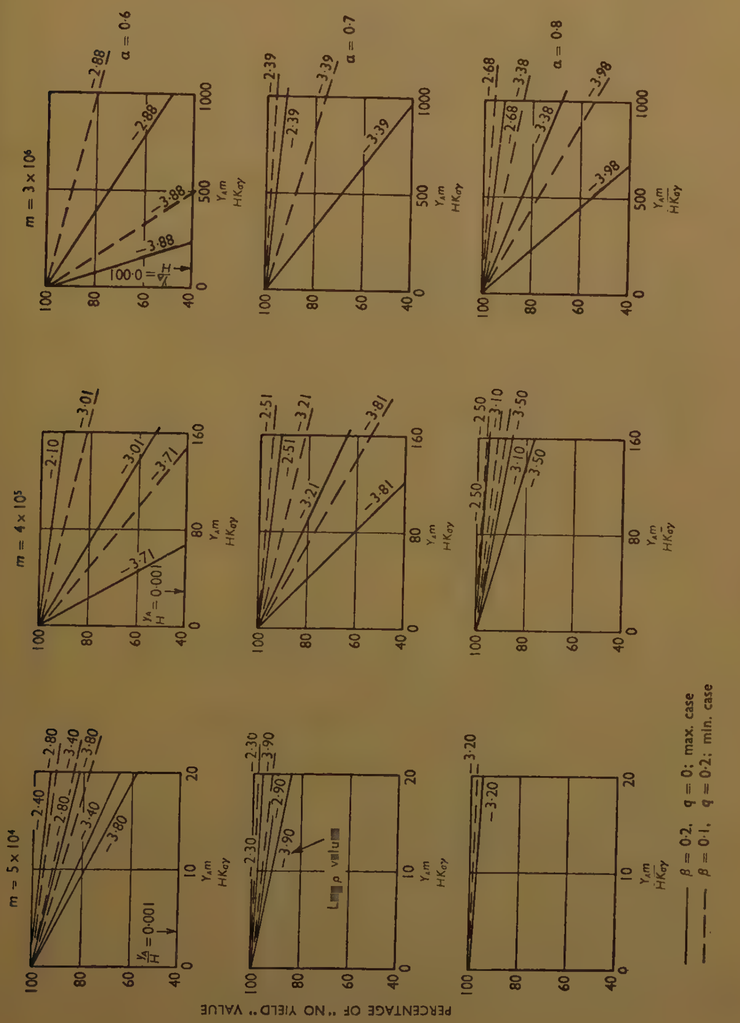
Fig. 12



Final average reduction curves for design use

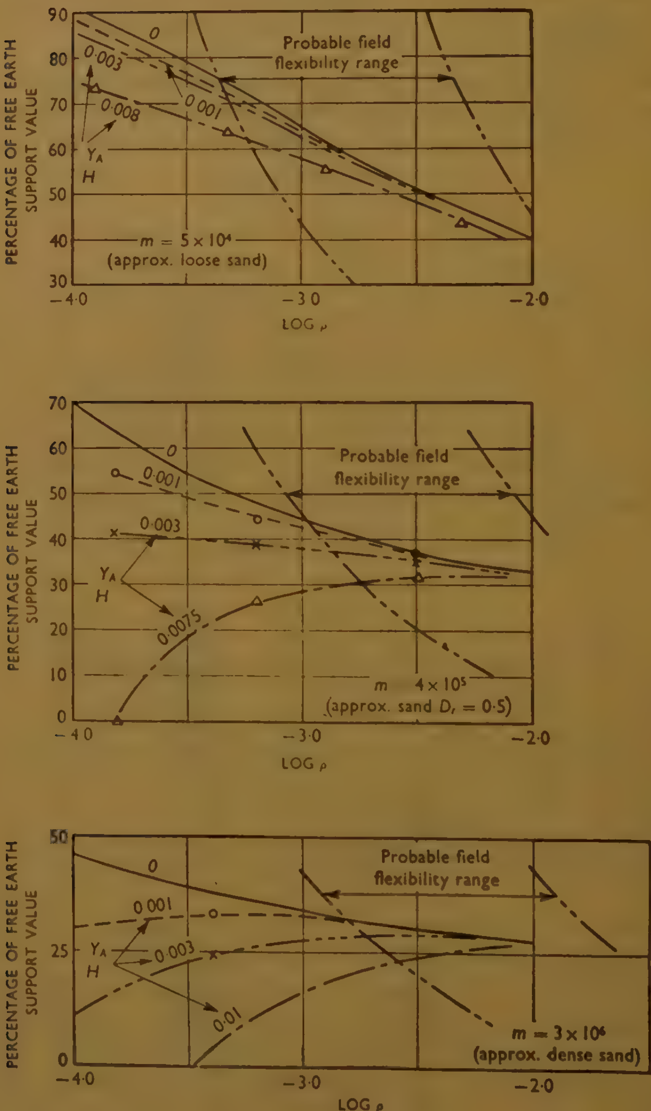
for design office use. Furthermore, since the percentage reduction curve is independent of the value of the active pressure coefficient $K_a\gamma$, the curve may be applied to the practical cases of variable soil strata and tidal conditions which are at present beyond exact individual mathematical analysis.

The final mean tie-rod load and bending-moment reduction curves are given in *Fig. 12*, and these curves summarize the results of the theoretical structures analysed. A final useful rule is that the reduction



TIE LOAD REDUCTIONS WITH ANCHORAGE YIELD

Fig. 14

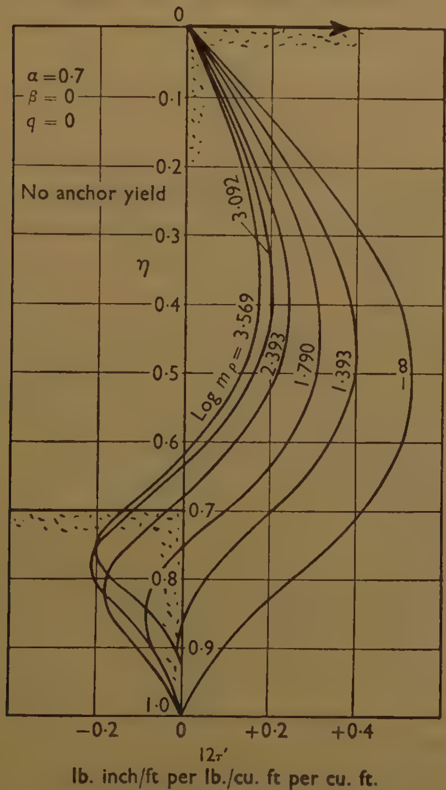


POSITIVE MOMENT REDUCTION WITH ANCHORAGE YIELD

in tie-rod load is approximately 60 per cent of the reduction in bending moment for any given subsoil and pile flexibility.

Final theoretical results of value are the influence of tie-rod yield on the maximum tie-rod loads and bending moments, *Figs 13* and *14*, and typical bending moment and pressure distributions, *Figs 15* and *16*.

Fig. 15



TYPICAL MOMENT DISTRIBUTION CURVES

Figs 13 and *14* show that the small anchor yields anticipated on actual structures for the practical flexibility range will only affect the tie loads and bending moments by a few per cent, so that the results for $\frac{Y_A}{H} = 0$ may be used for design.

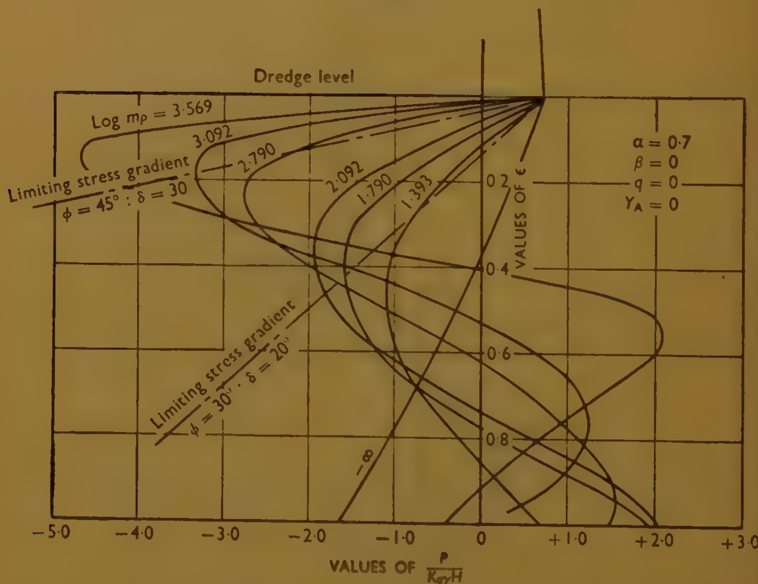
The theoretical curves, *Figs 10* and *11*, are of considerable value because they are not confined merely to the cases of loose and dense

sand or gravel, but may be extended to cases of silty subsoil or adverse seepage effects by a study of the value of the soil stiffness modulus, m.

Experimental Study of the Soil Stiffness Modulus m

The stress-strain curves for soil are not linear so that the modulus decreases with increase in the pressure/depth ratio and therefore with the degree of wall movement. However, the approximate value of the

Fig. 16



Typical pressure distributions

modulus for dense sand is about 20 times that for loose sand and about 100 times that for loose silt. Compared with such differences the curves of the observed moment/angular rotation relations for stiff walls may be replaced by average straight lines. Since the actual load coefficient for soil retained by sheet-pile walls has values within a limited range, it is possible to specify from observations and theory that the average angular movements of walls below the dredge level lie in the regions indicated in Table 1.

The rotations and deflexions of an actual flexible wall vary over the penetration depth so that the value of m must vary with depth. The degree of error resulting from the use of a constant value of m corre-

TABLE 1

Soil type	Average angular rotation
Dense sand	$0{\cdot}0005^c$
Loose sand	$0{\cdot}01^c$
Very loose silt	$0{\cdot}05^c$

sponding to average angular rotations may be examined from the results of the following methods of estimating the modulus.

(i) *Direct measurement of the moment/flexibility reduction curve*

Previous model sheet-pile tests were extended to include subsoils of: (*a*) sand, 90 per cent–mica, 10 per cent, loose density 86 lb./cubic foot; (*b*) loose silt from the Manchester Ship Canal, density 54 lb./cubic foot; and (*c*) coal fire ashes passing sieve No. 25, density 41 lb./cubic foot. The percentage reduction curves are shown in *Fig. 17* and also the theoretical curve for $\alpha = 0{\cdot}7$; *Fig. 11* has been fitted over each observed curve by means of tracing paper.

It is clear that the theoretical and observed curves fit very well with the more compressible subsoil, but that, with denser subsoils, the observed curves appear to twist clockwise relative to the theoretical. This continues until a wide divergence occurs for stiff walls in dense sand, whereas the results for flexible walls in dense sand fit the theoretical values quite well. These matters are explained by the following subsidiary effects:

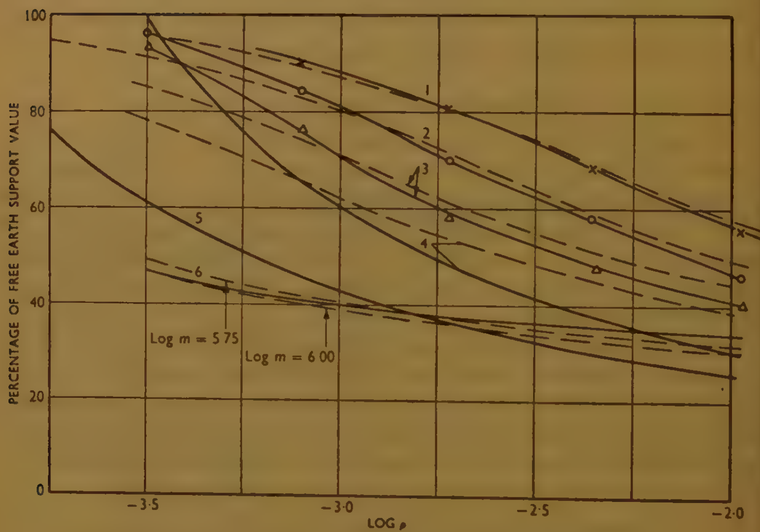
(*a*) Influence of outward deflexion of the wall above the dredge level on the value of $K_a\gamma$. For stiff walls in dense sand K_a is much larger. (See reference.[6])

(*b*) Reduction of active pressure at the dredge level due to subsoil restraint. This would be negligible with compressible subsoils which do not provide a rigid restraint.

(*c*) Poisson's ratio effect. A wall subject to a bending distribution as in *Fig. 15* (curve denoted by $\log mp = -3{\cdot}57$) suffers no moment at the tie and at a point just above the dredge level. Between these points large bending occurs, causing tension strains outside, and compression strains inside, the wall. These induce lateral compression and tension strains respectively, causing the wall to tend to curl concave outwards. At points of zero bending moment no curling occurs so that as these points become closer with increase in flexibility of the wall they exert a restraining influence on the curling at the point of maximum moment. The models used had been cut in vertical strips to prevent restraint from the wall on either side of the section so that restraint never occurred for this reason. Nevertheless, concrete walls grouted between

sections would provide lateral restraint, and the interlocks of steel piling would hardly be frictionless after driving. Taking Poisson's ratio as 0·3, the maximum stress reduction is 9 per cent which would appear as a bending-moment reduction in *Fig. 17*.

Fig. 17



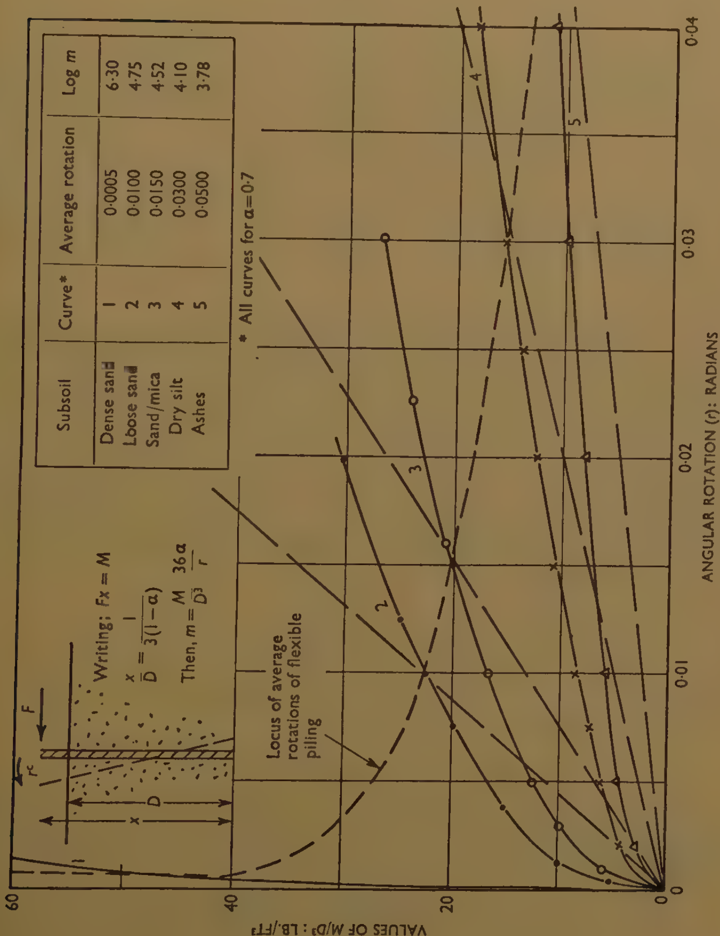
Subsoil	Curve	Experimental	Theoretical	Log m
Loose fine ashes	1	x—x—x	— — —	3·95
Loose dry silt	2	o—o—o	— — —	4·22
Loose sand/mica	3	Δ—Δ—Δ	— — —	4 52
Loose sand	4	Ref. 1, Fig. 14	— — —	4·75
Dense sand	5	Ref. 1, Fig. 14	———	—
Dense sand	6	Ref. 6, Fig. 36 (b)	— — —	5·75 – 6·00

Experimental and theoretical moment/flexibility curves

(*d*) The limiting pressure gradient at the dredge level. In the theoretical calculations no limit was placed on the maximum pressure gradient at the dredge level that the soil could withstand. *Fig. 16* shows that, taking $\log m = 4{\cdot}75$ for loose sand and 6·30 for dense sand, the limiting stress is exceeded near the dredge level in the practical flexibility range $\log \rho = -3{\cdot}2$ to $-2{\cdot}5$. However, a subsoil initially loose compacts in the highly stressed zone with shear (see Tschebotarioff [7]) and the limiting stress gradient will increase.

This would not occur with dense sand so that the theoretical moment reduction curve would tend to err to low values as flexibilities of the order $\log \rho = -2{\cdot}0$ are approached. No error occurs with compressible subsoils where $\log m = 4{\cdot}0$.

Fig. 18



FREELY EMBEDDED STIFF WALL ROTATION TESTS

Thus, since none of the influences (*a*) to (*d*) occur for highly compressible subsoil, agreement is expected with theory where moment reduction is entirely due to flexure. As a loose sand subsoil is approached (*Fig. 17*) the K_a values increase slightly at the stiff sections and effects (*b*) and (*c*) operate at the flexible sections. For dense sand the influence of (*d*)

counteracts (*b*) and (*c*) in the flexible range, and agreement in shape of curve results.

The fitting of the experimental and the theoretical curves in *Fig. 17* has been made with these influences in mind, but since the exact values of influences (*a*) to (*d*) have not been determined at each stage, an error must arise in the estimation of the value of log m. The values are shown in Table 2, column 1.

(ii) *Simple test on a freely embedded stiff wall*

A steel plate, preferably not less than 1 foot deep and 2 feet wide, may be buried in a sample of the subsoil and the graph of overturning moment about the toe plotted against the angular rotation of the wall, in radians (see *Fig. 18*). A rough value of m is obtained from the slope of the straight line through the origin and a point on each curve representing the average order of angular rotation expected in the field. The values obtained in this manner are shown in Table 2, column 2.

(iii) *From stress-strain theory and shear tests*

The distribution of pressure on a stiff freely embedded wall may be calculated using shear-box or triaxial compression stress-strain curves.[2] The calculation gives a pressure distribution close to the parabolic type of curve, and by fitting a parabola to the calculated distribution a value of m is obtained. The values are shown in Table 2, column 3.

For subsoils more compressible than loose sand the calculation need not be repeated in detail. The pressure at depth ϵD below the soil surface on a freely embedded pile, subject to a parabolic type pressure distribution, is given by the coefficient:

$$\frac{p}{\gamma D} = \frac{mr}{\gamma} \cdot \epsilon \cdot (\bar{\epsilon} - \epsilon)$$

where $\bar{\epsilon}D$ denotes depth to the point of rotation.

It was shown [2] that,

$$r(\bar{\epsilon} - \epsilon) = \epsilon_s \tan\left(45 - \frac{\phi}{2}\right) \cdot f(\epsilon)$$

so that, writing, $D = (1 - \alpha)H,$

$$\frac{p}{\gamma H} = (1 - \alpha)\epsilon f(\epsilon) \cdot \frac{m}{\gamma} \epsilon_s \cdot \tan\left(45 - \frac{\phi}{2}\right) \quad . \quad . \quad . \quad (34)$$

For a given degree of ϕ mobilized at depth ϵD, $\frac{p}{\gamma D}$ is dependent on ϕ only and will be constant for varying values of ϵ_s and γ.

Hence, $\frac{m\epsilon_s}{\gamma} = \text{constant}.$

Therefore, taking loose sand as a standard,

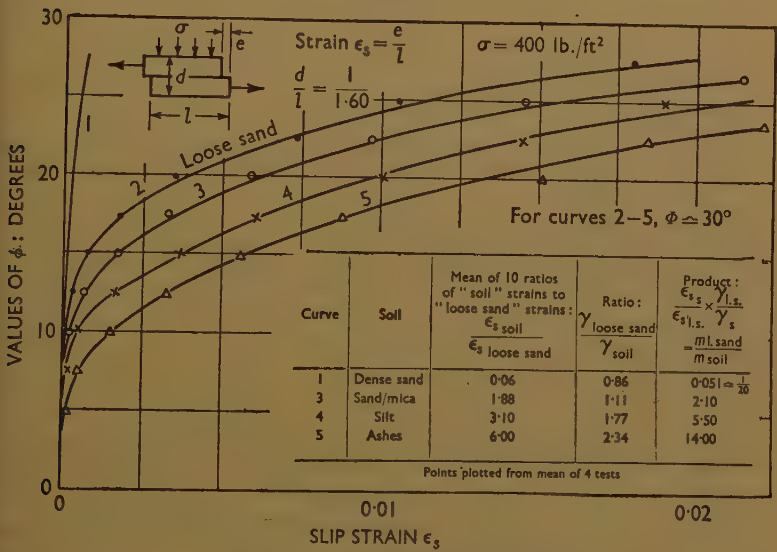
$$\log m_r = \log \frac{m_{\textit{loose sand}}}{m_{\textit{subsoil}}}$$

$$= \log \frac{\epsilon_{s\ \textit{subsoil}}}{\epsilon_{s\ \textit{sand}}} \times \frac{\gamma_{\textit{sand}}}{\gamma_{\textit{subsoil}}}$$

and $$\log m_{\textit{subsoil}} = 4{\cdot}75 - \log m_r.$$

The stress-strain curves of *Fig. 19* were drawn for a confining pressure equivalent to the mean penetration depth of a 40-foot pile, namely 6 feet of immersed sand or 4 feet of dry sand. The ratios of the strains were

Fig. 19



Curve	Soil	Mean of 10 ratios of "soil" strains to "loose sand" strains: $\frac{\epsilon_{s\ soil}}{\epsilon_{s\ loose\ sand}}$	Ratio: $\frac{\gamma_{loose\ sand}}{\gamma_{soil}}$	Product: $\frac{\epsilon_s}{\epsilon_{s\,l.s.}} \times \frac{\gamma_{l.s.}}{\gamma_s} = \frac{m_{l.\,sand}}{m_{soil}}$
1	Dense sand	0·06	0·86	0·051 ≃ $\frac{1}{20}$
3	Sand/mica	1·88	1·11	2·10
4	Silt	3·10	1·77	5·50
5	Ashes	6·00	2·34	14·00

SHEAR-BOX STRESS-STRAIN CURVES

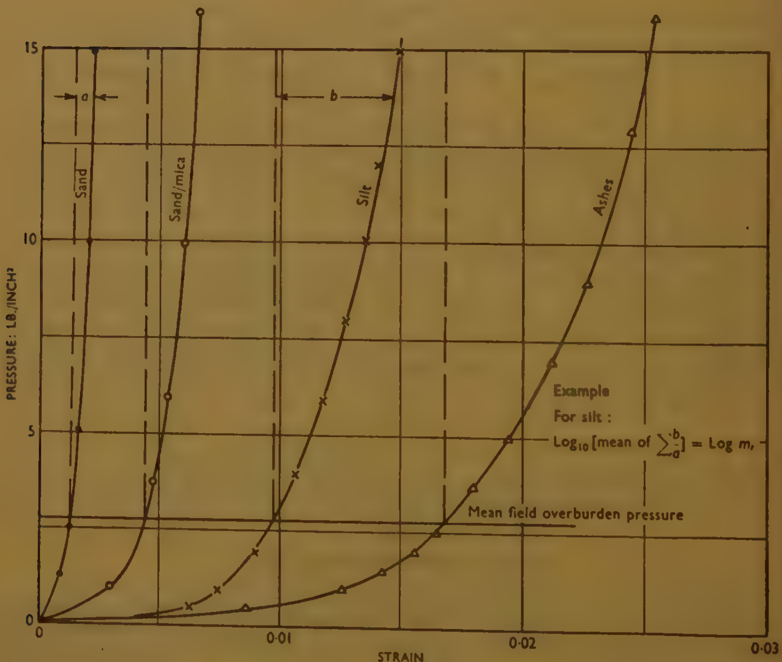
calculated for ten values of ϕ from $7\frac{1}{2}$ degrees to 30 degrees at $2\frac{1}{2}$-degree intervals, using mean results from four repeated tests in each case. The mean ratio in each case is given in *Fig. 19* and was used to calculate the log m values shown in Table 2, column 3 (*a*).

(iv) *Confined compression test*

The confined compression test applies partly pure compressive strain and partly shear strain to the soil. The stress system acting on the subsoil in front of a flexible wall also results in compressive and shear strains, but of entirely different relative values. Nevertheless, the ratio of the "confined compression strain increases" for a particular pressure range for a compressible subsoil to that of loose sand might be expected to be

independent of the stress system and depend only on the ratio of the soil stiffness properties. For example, the ratio of the deflexions of a point in a steel structure and a similar aluminium structure would be the inverse ratio of the moduli of the two materials, a ratio which could then be applied to the two frames under a completely different stress system. The initial average field confining pressure for a 40-foot backfilled pile is indicated on the stress-strain curves in *Fig. 20* and the ratios of strain increases above this value are reasonably constant within the expected field pressure range. Assuming these ratios give a measure of the inverse ratio of the m values, taking $\log m = 4{\cdot}75$ for loose sand, the values of

Fig. 20



CONFINED COMPRESSION CURVES

log m are easily calculated and are shown in Table 2, column 4. This is an extension of previous reasoning [1] where the logarithm of the initial slope of the compressibility curve was found to be proportional to the logarithm of the critical flexibility number. It is now seen that the critical flexibility number was an arbitrary measure of the soil stiffness modulus.

Examination of all the values in Table 2 shows that, with the exception of dense sand, where variations in log m have little influence on

TABLE 2

Soil type	Values of $\log_{10} m$					
	1	2	3	3 (a)	4	Mean
Dense sand	5·75–6·00	6·30	6·35	6·05	—	6·10
Loose sand	4·75	4·75	4·75	—	—	4·75
90% sand, 10% mica	4·52	4·52	—	4·42	4·36	4·45
Loose dry silt	4·22	4·10	—	4·01	3·98	4·08
Loose fine ashes	3·95	3·78	—	3·60	3·75	3·77

bending moment, etc. (see *Fig. 17*), the values of $\log m$ agree to $\pm 0{\cdot}18$ on the mean values. This is equivalent to $\pm 0{\cdot}14$ on an estimation of relative density. This range of error is well within the likely range of error in sampling and estimating the average properties of variable soil strata.

The figures in columns 3 and 4 correspond to average field pressures, whereas those in columns 1 and 2 correspond to model pressures. It is to be expected that the value of $\log m$ will decrease slightly with increase in confining pressure. The reduction curves of *Fig. 17* corresponding to model tests would require to be moved slightly to the right. The exact extent of the variation cannot at present be calculated since it is not yet possible to obtain accurate stress-strain curves at confining pressures acting on models. The work by Tschebotarioff, at three times the scale, showed agreement with the position of the reduction curve for loose sand, but since the subsoil in those tests was immersed in water, the confining pressures were only about double those used by the Author. Observations on actual piling are never likely to be coupled with sufficiently accurate data concerning the subsoil state to yield even a rough estimation of the influence of increase in confining pressure.

The practical task of surveying cohesionless subsoil below water level for sheet-piling works can at present be accomplished only by some form of penetration test. Numerous types of test and equipment exist, but whichever is available may be calibrated by driving into artificial beds of dense sand, loose sand, and loose silt or ashes, respectively, which are about 6 feet deep. Provided a check is made on the compression curves for the silt or ashes used, the mean values shown in Table 2 may be taken as standards to calibrate the equipment.

A knowledge of the $\log m$ value allows a more accurate design of cantilever walls. The results of model tests with cantilever walls plotted in *Fig. 5* show good agreement with theory.

Influence of Seepage

Upflowing water through the subsoil reduces the effective soil density. To maintain the same passive resistance to a given overturning moment the wall movement increases to shear the soil further until a higher value of ϕ is mobilized. Thus, at a first approximation, the value of m is proportional to the effective soil density :

$$\log m_r = \frac{\log m_{\text{loose sand}}}{\log m_{\text{seepage}}} = \frac{\log \gamma_{\text{loose sand}}}{\gamma_{\text{seepage}}} \quad . \quad . \quad . \quad (35)$$

Loose sand subsoil immersed in static water has also a reduced log m value over that of dry sand. However, if the water level is near the top of the piling, the active soil value γ is also reduced to the same degree and the value of γ does not influence the moment reduction curve. For this case, therefore, can be estimated the log m_r value due to seepage relative to the immersed subsoil :

$$\log m_r = \log \frac{\gamma_{\text{immersed}}}{\gamma_{\text{seepage}}}.$$

From simple piping theory the effective soil density is :

$$\gamma_{\text{seepage}} = \gamma_{\text{immersed}} - \frac{h_a}{D}\gamma_w$$

where ha denotes excess hydrostatic head at toe of pile.

Therefore,
$$\log m_r = \log \frac{1}{1 - \frac{h_a}{D} \cdot \frac{\gamma_w}{\gamma_i}}$$

And, since $\gamma_w \simeq \gamma_i$,

$$\log m_r \simeq \log \frac{1}{1 - \frac{h_a}{D}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

Simple tests were carried out using a 9-inch-deep stiff plate, 18 inches wide, rotating about the toe (*Fig. 21* (*a*)). Water pressure was applied at the toe, and the pressure-head at three points up the wall was measured by manometers. The wall was sealed at the corners by thin surgical rubber. The apparatus was first calibrated by observing the overturning moment M_s and angular rotation r for the case of static water to the top of the wall and side friction due to the rubber seal. The wall was then backfilled with a uniformly graded sand and gravel in the loose state and a total overturning moment M_t was applied. After the wall rotation had reached a steady state, excess water pressure was applied at the toe in increments. The additional water pressure on the wall was plotted from the manometer readings and an additional moment δM_t was applied to the wall to counterbalance this pressure at each increment. The final rotations were recorded.

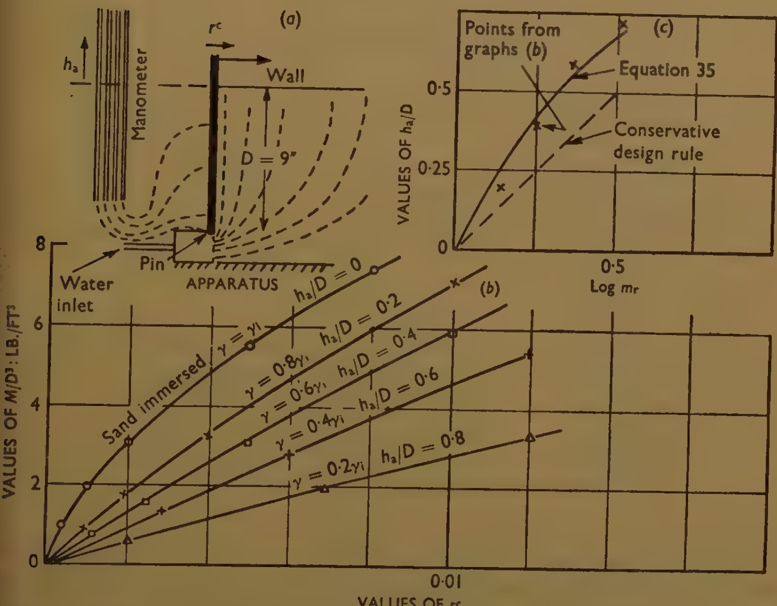
Taking $K_0 = 0{\cdot}4$, the " at rest " overturning moment for each value of the effective soil density was calculated, M_{ar}.

The resistance to movement of the wall by the soil is given by:

$$(M_t + \delta M_t) - M_s - M_{ar} - \delta M_t = M.$$

These values are plotted in *Fig. 21* (*b*) and the mean ratios of the abscissae of the curves are plotted against the ratio $\dfrac{h_a}{D}$ in *Fig. 21* (*c*). Equation (35) is also plotted on this diagram.

Fig. 21



PINNED WALL MOMENT/ROTATION CURVES

INFLUENCE OF SEEPAGE ON THE SOIL STIFFNESS MODULUS

The equation (36) assumes a uniform uplift throughout the whole lepth of penetration of the piling, whereas the actual uplift decreases ccording to the flow net diagram. However, a decrease in density of he subsoil under a constant moment M requires that the K_p values, that s to say, the ϕ values, increase; these are coupled with strain values rger than simple proportion as assumed in equation (35). These two nfluences have opposite effects and this may be the reason why the mple curve and observed readings agree quite well in *Fig. 21* (*c*).

Nevertheless, pending a fuller investigation of the influence of seepage, it would be conservative to allow the dotted line given by:

$$\log m_r = \frac{h_a}{D},$$

where

$$\frac{h_a}{D} < 0{\cdot}5.$$

For example, if a differential water level of θH is allowed on the design, (*Fig. 22*), then head at toe:

$$h_a \simeq \theta H\left[1 - \frac{H}{(2-\alpha)H}\right]$$

and

$$\frac{h_a}{D} = \frac{h_a}{(1-\alpha)H} = \frac{\theta}{(2-\alpha)}.$$

For $H = 30$ feet, $\theta H = 3$ feet, $\alpha = 0{\cdot}7$, $\dfrac{h_a'}{D} = 0{\cdot}06 \simeq \log m_r$.

Fig. 22

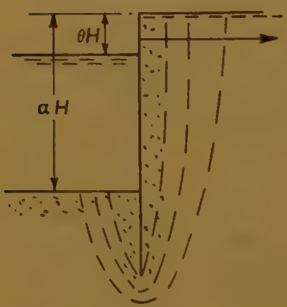
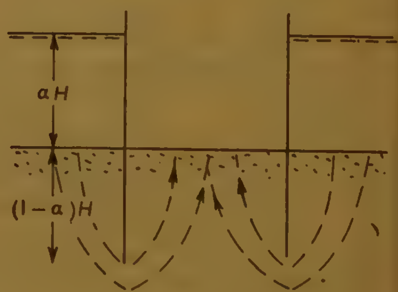


Fig. 23



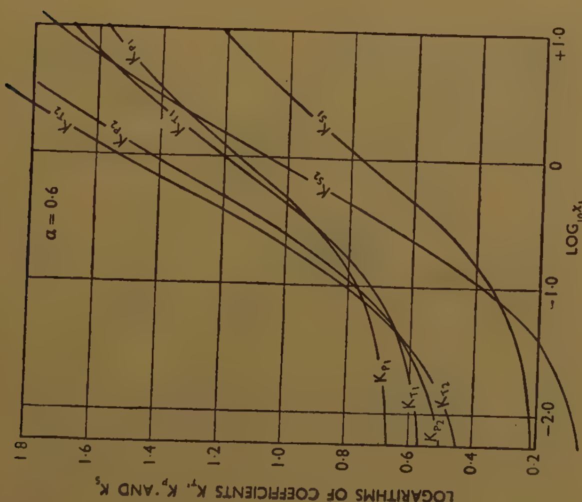
The use of the reduced shear strength of the subsoil would lead to a lower value of α which causes a negative value of $\log m_r$ (see *Fig. 11*), so that for the usual order of differential pressures allowed, no account need be taken of the influence of seepage on the bending moment on the wall. On the other hand, for the case of a cofferdam (*Fig. 23*):

$$\frac{h_a}{D} = \frac{\alpha}{2(1-\alpha)}$$

If

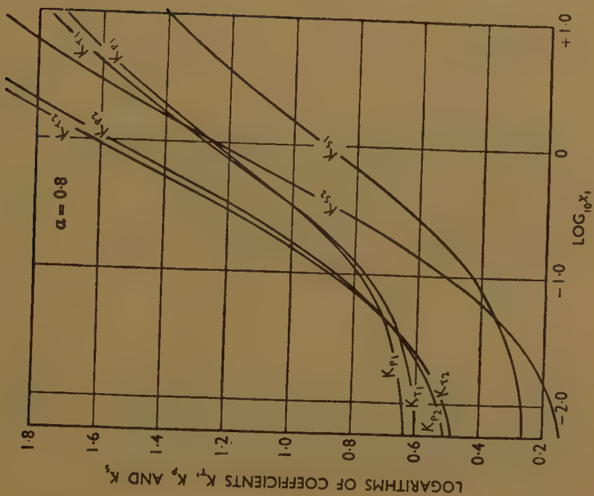
$$\alpha = 0{\cdot}5, \quad \frac{h_a}{D} = 0{\cdot}5.$$

In addition, the large extra active pressure due to the water will cause an additional $\log m_r$ increment, approximately 0·2 (see reference 6, *Fig. 36* (*a*), curve C—curve B). Hence the total shift of the moment/flexibility curve would be 0·7. This would more than offset the decrease in $\log m_r$ due to decrease in design value of α.



α = 0·6
LOGARITHMS OF COEFFICIENTS K_T, K_P AND K_S
$LOG_{10}X_1$

Fig. 24



α = 0·8
LOGARITHMS OF COEFFICIENTS K_T, K_P AND K_S
$LOG_{10}X_1$

Fig. 25

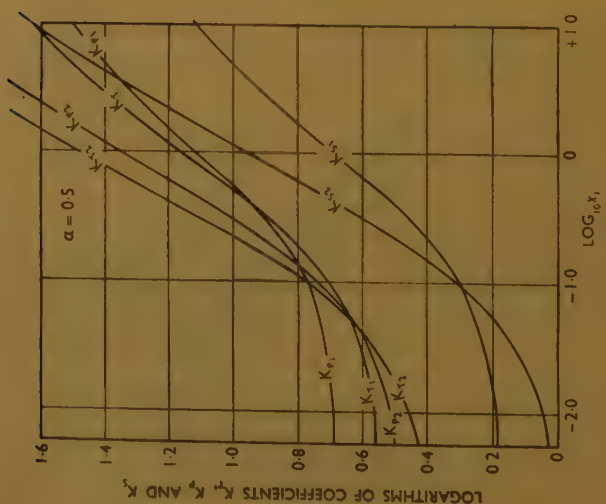

α = 0·5
LOGARITHMS OF COEFFICIENTS K_T, K_P AND K_S
$LOG_{10}X_1$

Fig. 26

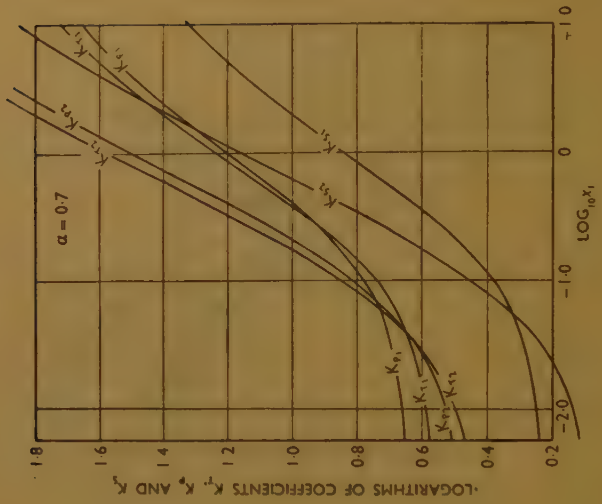

α = 0·7
LOGARITHMS OF COEFFICIENTS K_T, K_P AND K_S
$LOG_{10}X_1$

Fig. 27

Summary

The mathematical analysis of sheet-pile walls based on the simplifying assumption that the subsoil passive pressure varies linearly with depth and deflexion leads to moment/flexibility curves in close agreement with those observed. The position of the moment/flexibility curve on the $\log \rho$ axis is determined by the value of the soil stiffness modulus. This may be estimated by comparing the records of penetration tests in the actual soil with that in artificial beds of soil in a laboratory. Seepage may increase the bending moment on cofferdam walls by decreasing fixity.

The results are presented in graphs which are readily applicable to design office use in conjunction with the free earth support analysis. Previous work is extended to cover cantilever piling and all ranges of cohesionless subsoils.

Acknowledgements

The work was carried out in the Engineering Department of Manchester University. The Author is indebted to Professor J. A. L. Matheson, M.B.E., Ph.D., M.Sc., M.I.C.E., for providing the facilities for the work.

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The Paper is accompanied by twenty-four sheets of diagrams, from which the Figures in the text have been prepared, and by the following Appendix.

CORRESPONDENCE on this Paper should be received at the Institution before the 15th May, 1955, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

APPENDIX

$$f_1 = 1 - 36\chi + 13{\cdot}71428571\chi^2 - 0{\cdot}712430426\chi^3 + 0{\cdot}009925501\chi^4$$

$$f_2 = 1 - 8\chi + 1{\cdot}870129870\chi^2 - 0{\cdot}071928072\chi^3 + 0{\cdot}000805917\chi^4$$

$$f_3 = 1 - 2{\cdot}857142857\chi + 0{\cdot}432900432\chi^2 - 0{\cdot}012592450\chi^3 + 0{\cdot}000114763\chi^4$$

$$f_4 = 1 - 1{\cdot}285714285\chi + 0{\cdot}131868132\chi^2 - 0{\cdot}002955028\chi^3 + 0{\cdot}000022130\chi^4$$

$$f_5 = 1 - 0{\cdot}666666667\chi + 0{\cdot}047952048\chi^2 - 0{\cdot}000841264\chi^3 + 0{\cdot}000005225\chi^4$$

$$K_{T_1} = \frac{\begin{aligned}(3+\alpha) &+ \chi_1[10{\cdot}666666667 + 19{\cdot}333333333\alpha] \\ &+ \chi_1^2[3{\cdot}532467542 + 10{\cdot}85314676\alpha] \\ &+ \chi_1^3[0{\cdot}272569532 + 1{\cdot}154574748\alpha] \\ &+ \chi_1^4[0{\cdot}007276387 + 0{\cdot}038828750\alpha] \\ &+ \chi_1^5[0{\cdot}000006919 + 0{\cdot}000821481\alpha]\end{aligned}}{\text{Divisor}}$$

$$K_{T_2} = \frac{\begin{aligned}(2+\alpha) &+ \chi_1[15{\cdot}0 + 46{\cdot}714285715\alpha] \\ &+ \chi_1^2[7{\cdot}192807021 + 37{\cdot}378620719\alpha] \\ &+ \chi_1^3[0{\cdot}713572190 + 5{\cdot}069686870\alpha] \\ &+ \chi_1^4[0{\cdot}023052547 + 0{\cdot}205234052\alpha] \\ &+ \chi_1^5[0{\cdot}000231627 + 0{\cdot}003533983\alpha]\end{aligned}}{\text{Divisor}}$$

$$K_{P_1} = \frac{\begin{aligned}(6 - 3\alpha + \alpha^2) &+ \chi_1[11{\cdot}714285714 + 8{\cdot}571428571\alpha + 9{\cdot}714285715\alpha^2] \\ &+ \chi_1^2[2{\cdot}765234748 + 5{\cdot}066933126\alpha + 6{\cdot}553446426\alpha^2] \\ &+ \chi_1^3[0{\cdot}168480792 + 0{\cdot}480747006\alpha + 0{\cdot}777916482\alpha^2] \\ &+ \chi_1^4[0{\cdot}003749208 + 0{\cdot}014330745\alpha + 0{\cdot}028025179\alpha^2] \\ &+ \chi_1^5[0{\cdot}000009138 + 0{\cdot}000002481\alpha + 0{\cdot}000816781\alpha^2]\end{aligned}}{\text{Divisor}}$$

$$K_{P_2} = \frac{\begin{aligned}3 &+ \chi_1[11{\cdot}142857145 + 22{\cdot}714285723\alpha + 27{\cdot}857142887\alpha^2] \\ &+ \chi_1^2[3{\cdot}683744838 + 14{\cdot}210931387\alpha + 26{\cdot}676751518\alpha^2] \\ &+ \chi_1^3[0{\cdot}284287173 + 1{\cdot}572142224\alpha + 3{\cdot}926829657\alpha^2] \\ &+ \chi_1^4[0{\cdot}007589058 + 0{\cdot}053979525\alpha + 0{\cdot}166718016\alpha^2] \\ &+ \chi_1^5[0{\cdot}000062535 + 0{\cdot}000569811\alpha + 0{\cdot}003137250\alpha^2]\end{aligned}}{\text{Divisor}}$$

$$K_{S_1} = \frac{\begin{aligned}(1+\alpha) &+ \chi_1[1{\cdot}952380953 + 6{\cdot}761904762\alpha + 6{\cdot}285714286\alpha^2] \\ &+ \chi_1^2[0{\cdot}460872458 + 2{\cdot}610722618\alpha + 4{\cdot}121212109\alpha^2] \\ &+ \chi_1^3[0{\cdot}028080132 + 0{\cdot}216409268\alpha + 0{\cdot}469083034\alpha^2] \\ &+ \chi_1^4[0{\cdot}000624868 + 0{\cdot}006026651\alpha + 0{\cdot}016401047\alpha^2] \\ &+ \chi_1^5[0{\cdot}000001523 + 0{\cdot}000003873\alpha + 0{\cdot}000408804\alpha^2]\end{aligned}}{\text{Divisor}}$$

$$K_{S_2} = \frac{\begin{aligned}(0{\cdot}5 + \alpha) &+ \chi_1 [1{\cdot}857142857 + 11{\cdot}285714286\alpha + 17{\cdot}714285714\alpha^2] \\ &+ \chi_1^2[0{\cdot}613957476 + 5{\cdot}964892066\alpha + 15{\cdot}706864744\alpha^2] \\ &+ \chi_1^3[0{\cdot}047381195 + 0{\cdot}618809808\alpha + 2{\cdot}225438526\alpha^2] \\ &+ \chi_1^4[0{\cdot}001264843 + 0{\cdot}020522861\alpha + 0{\cdot}092355591\alpha^2] \\ &+ \chi_1^5[0{\cdot}000010422 + 0{\cdot}000210782\alpha + 0{\cdot}001663593\alpha^2]\end{aligned}}{\text{Divisor}}$$

$$\begin{aligned}\text{Divisor} = 1 &+ 1{\cdot}142857130\chi_1 + 0{\cdot}197802211\chi_1^2 + 0{\cdot}009642718\chi_1^3 \\ &+ 0{\cdot}000180303\chi_1^4 + 0{\cdot}000000247\chi_1^5\end{aligned}$$

Paper No. 5990

"Sheet-Pile Walls Encastré at the Anchorage"

by

* Peter Walter Rowe, Ph.D., A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

Previous experimental [1] and theoretical [2] analyses of sheet-pile walls anchored by elastic tie-rods are extended to include walls built in at the top to a concrete relieving platform. Theoretical reduction curves for the anchor force and maximum positive and negative bending moments, from flexure alone, are given. Agreement with model observations is obtained, allowing for further moment reduction from arching of the retained soil.

INTRODUCTION

A WALL built in to a concrete platform cannot yield outwards at the anchorage. Therefore, moment reduction from arching of the backfill will operate together with that arising from flexure. It is proposed, however, to calculate the variation in moment on the wall with flexure, assuming a triangular active pressure distribution, and then to apply further reduction resulting from arching.

Since a relieving platform transfers surcharge loading to the subsoil by means of bearing piles, surcharge is not included in the calculation. All walls are treated as being anchored at the top, the height of the wall being taken from the underside of the platform to the toe (see *Fig. 1*).

For the analysis of the wall pinned at the anchorage,[2] the wall was separated into three cantilevered wall systems, namely :—

(1) Cantilever, subject to a line-load at the top.
(2) Cantilever, subject to a triangular pressure distribution.
(3) Cantilever, subject to a uniform pressure distribution.

Case (3) is not required here since no surcharge occurs. The values of the deflexions and slopes of the walls at the dredge level for cases (1) and (2) are required again for the following analysis.

The solution to the pinned wall was obtained by equating the difference between the outward deflexions of the wall, in cases (1) and (2), to the

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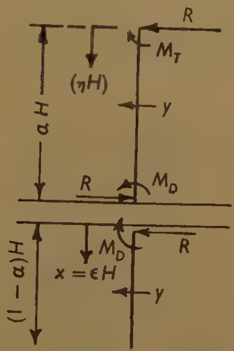
* The Author is Senior Lecturer in Engineering at the University of Manchester.
[1] The references are given on p. 86.

yield of the anchorage. In this case, no yield occurs so that the deflexions of cases (1) and (2) are made equal. In addition, it is necessary to apply a moment at the top of the wall to produce a slope of the wall equal, and opposite, to that of the pinned wall. The wall is then encastré, and the moment is the fixing moment.

Fig. 1

Fig. 2



The notation and method of analysis follow that of the pinned pile, so that it is only necessary to present the additional notation and the case of the cantilever pile subject to a bending moment and shear force at the top.

Additional Notation

a_1	denotes slope at dredge level.
a_0	,, deflexion at dredge level.
KM_1, KM_2	denote functions of X_1 and α.
Km_1, $K'm_1$	,, functions of χ_1.
Km_2, $K'm_2$	,, functions of χ_1.
M_T	denotes the fixing moment at the top of the wall.
M_D	,, moment at the dredge level due to M_T and R.
R	,, additional anchor force due to encastré condition.

Solution

1) *Cantilevered pile subject to moment:*

M_T and line-load R at the top (*Fig.* 2).

Considering that part of the pile above the dredge level :—

$$EI\frac{d^2y}{d(\eta H)^2} = -M_T + \frac{\eta}{\alpha}(M_T - M_D)$$

$$\therefore EI\frac{dy}{d(\eta H)} = -M_T(\eta H) + \frac{\eta^2 H}{2\alpha}(M_T - M_D) + EI\,.\,\frac{dy}{d(\eta H)}_{(\eta=0)}$$

When
$$\eta = \alpha,\ \frac{dy}{d(\eta H)} = a_1$$

$$\therefore EI\frac{dy}{d(\eta H)}_{(\eta=0)} = EIa_1 + (M_T + M_D)\frac{\alpha H}{2} \quad . \quad . \quad . \quad (1)$$

$$EIy = -M_T\frac{(\eta H)^2}{2} + \frac{(\eta H)^3}{6\alpha H}\,.\,(M_T - M_D) + EIa_1\eta H + (M_T + M_D)\frac{\alpha\eta H^2}{2}.$$

Since $y=a_0$ when $\eta=\alpha$

$$EI\left[\frac{a_0}{(\alpha H)^2} - \frac{a_1}{(\alpha H)}\right] = \frac{1}{6}[M \quad + 2M_D] \quad . \quad . \quad . \quad . \quad (2)$$

It is now necessary to determine the values of a_0 and a_1 resulting from the application of a positive shear force and negative moment to the lower part of the pile embedded in the subsoil.

Following the previous analysis, the deflexion is expanded in a series of x,

$$y = a_0 + a_1x + a_2x^2 + a_3x^3 \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and
$$EI\frac{d^4y}{dx^4} = p = p_a - \frac{mxy}{(1-\alpha)H} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

At the dredge level, $x=0$, from *Fig. 2*,

$$\left.\begin{aligned}
&EI\frac{d^2y}{dx^2} = 2a_2EI = -M_D \\
\text{and}\quad &a_2 = -\frac{{}^{T}M_D}{2EI} \\
\text{also}\quad &EI\frac{d^3y}{dx^3} = 6a_3EI = R \\
&\therefore a_3 = \frac{R}{6EI} \\
\text{and}\quad &EI\frac{d^4y}{dx^4} = 4\,!\,a_4 = 0 \\
&\therefore a_4 = 0
\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

At the toe
$$x = (1-\alpha)H$$

and
$$\frac{d^2y}{dx^2} = \frac{d^3y}{dx^3} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Equation (3) is differentiated four times and the value of $\frac{d^4y}{dx^4}$ is equated

to that in equation (4). This leaves five unknowns $a_0, a_1 \ldots a_4$. The values of a_2, a_3, a_4 are given by equations (5), and the remaining unknown values a_0 and a_1 are obtained from the solutions of equations (6).

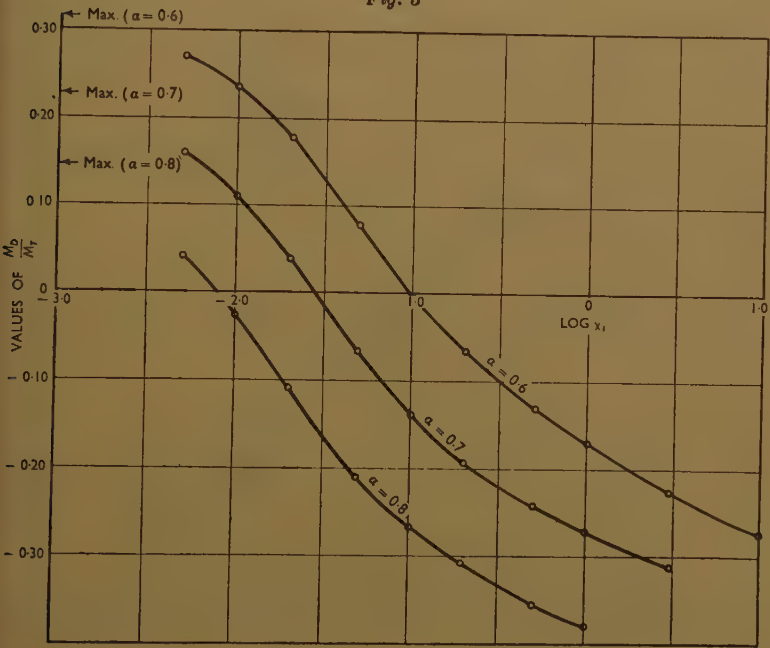
Whence : $$a_0 = \frac{-6}{m(1-\alpha)^2H^2}\{4M_D \,.\, Km_1 - (1-\alpha)RHK'm_1\}. \quad . \quad (7)$$

and $$a_1 = \frac{+12}{m(1-\alpha)^3H^3}\{3M_DKm_2 - (1-\alpha)RHK'm_2\} \quad . \quad . \quad (8)$$

Values of Km_1 $K'm_1$ Km_2 and $K'm_2$ are given in the Appendix and in *Fig. 14.*

By substituting the value $\dfrac{M_T - M_D}{\alpha H} = R$ in equations (7) and (8), a_0 and a_1 are obtained in terms of M_D and M_T. Substituting these values

Fig. 3



or a_0 and a_1 in equation (2) gives M_D as a ratio of M_T :

$$\frac{M_D}{M_T} = \frac{(1-\alpha)^2K'm_1 + 2\alpha(1-\alpha)K'm_2 - 4mp\alpha^3(1-\alpha)^3}{4\alpha(1-\alpha)Km_1 + (1-\alpha)^2K'm_1 + 6\alpha^2Km_2 + 2\alpha(1-\alpha)K'm_2 + 8mp\alpha^3(1-\alpha)^3} \quad (9)$$

where
$$\rho = \frac{1}{144} \frac{H^4}{EI}$$

For an infinitely stiff pile, equation (9) gives $\dfrac{M_D}{M_T} = \dfrac{3 - 2\alpha - \alpha^2}{3 + 2\alpha + \alpha^2}$ and, for an infinitely flexible pile, $\dfrac{M_D}{M_T} = -\dfrac{1}{2}$ (that is, encastré at the dredge level).

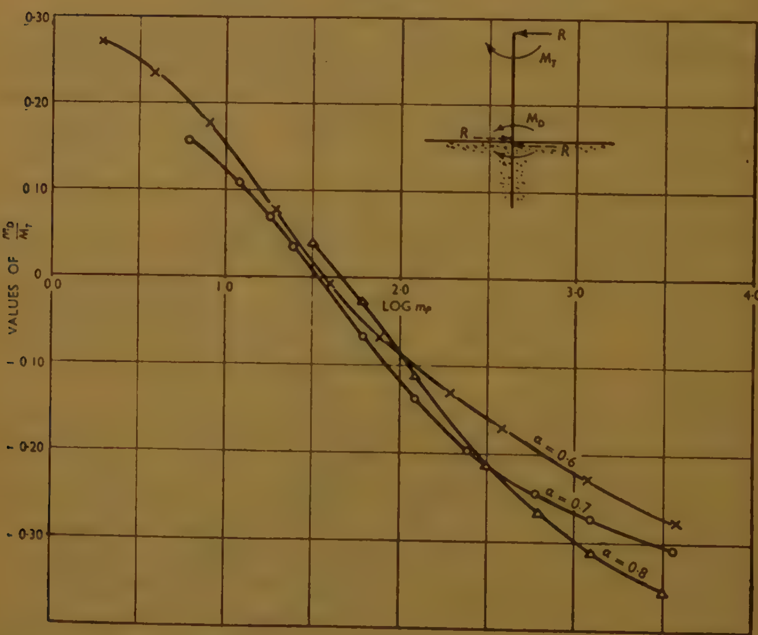
Substituting:

$$KM_1 = (1 - \alpha)^2 K'm_1 + 2\alpha(1 - \alpha)K'm_2$$

$$KM_2 = 4\alpha(1 - \alpha)Km_1 + (1 - \alpha)^2 K'm_1 + 6\alpha^2 Km_2 + 2\alpha(1 - \alpha)K'm_2$$

and
$$\chi_1 = \tfrac{1}{10}(1 - \alpha)^4 m\rho$$

Fig. 4



Equation (9) becomes:

$$\frac{M_D}{M_T} = \frac{KM_1 - \dfrac{40\chi_1\alpha^3}{(1 - \alpha)}}{KM_2 + \dfrac{80\chi_1\alpha^3}{(1 - \alpha)}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Values of KM_1 and KM_2 are given in the Appendix and in *Fig. 15.*

Ratios $\dfrac{M_D}{M_T}$ are shown in *Figs 3* and *4* for $\alpha = 0{\cdot}6 \rightarrow 0{\cdot}8$. The slope at the top of the pile is obtained by substituting the value of a_1 from equation (8) in equation (1), whence :

$$\frac{dy}{d(\eta H)_{\eta=0}} =$$

$$\frac{12M_T}{\alpha(1-\alpha)^3 mH^3}\left[\frac{M_D}{M_T}\{3\alpha Km_2 + (1-\alpha)K'm_2 + 6\alpha^2(1-\alpha)^3 mp\} - (1-\alpha)K'm_2 + 6\alpha^2(1-\alpha)^3 mp\right] \quad . \quad . \quad (11)$$

Substituting the value of $\dfrac{M_D}{M_T}$ from equation (10) in equation (11) gives the slope at the top of the pile arising from the application of moment, M_T, at the top.

It is now necessary to state the natural slope at the top of a pinned pile arising from the triangular active pressure, and line-load, T, at the top, respectively.

(2) *Pinned Anchored Pile:* $\beta = 0, \quad q = 0$

(*a*) Due to earth pressure on cantilever pile,

$$\text{Slope at dredge level} = K_a\gamma \, . \frac{2\alpha Kp_2}{m(1-\alpha)^3}$$

(equation 20, reference 2).

The additional slope from bending above the dredge level

$$= K_a\gamma \, . \frac{\alpha^4}{24} \, . \frac{H^4}{EI}$$

$$\therefore \left[\frac{dy}{d(\eta H)}\right]_{\eta=0} = K_a\gamma\left[\frac{2\alpha Kp_2}{m(1-\alpha)^3} + \frac{\alpha^4 \, . \, H^4}{24EI}\right] \quad . \quad . \quad (12)$$

Similarly for (*b*) cantilever pile with line-load, T

$$\left[\frac{dy}{d(\eta H)}\right]_{\eta=0} = \frac{T}{H_2}\left[\frac{12K_{T_2}}{m(1-\alpha)^3} + \frac{\alpha^2}{2} \, . \, \frac{H^4}{EI}\right] \quad . \quad . \quad (13)$$

Subtracting equation (12) from equation (13) gives the slope at the top of the pinned pile :

$$\left[\frac{dy}{d(\eta H)}\right]_{\eta=0}^{\text{pinned}} =$$

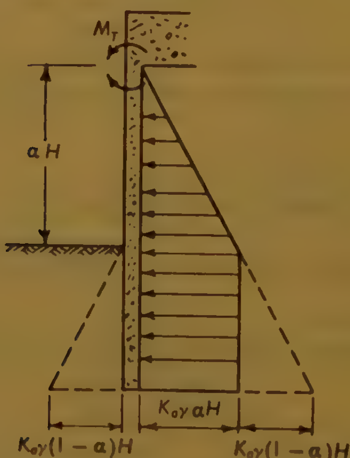
$$K_a\gamma\left\{6\alpha^2 p\left[12\frac{T}{K_a\gamma H^2} - \alpha^2\right] + \frac{2}{m(1-\alpha)^3}\left[6K_{T_2} \times \frac{T}{K_a\gamma H^2} - \alpha Kp_2\right]\right\} \quad . \quad . \quad (14)$$

Equating equations (11) and (14), the fixing moment M_T at the anchorage required to reduce the slope there to zero is obtained :

$$\frac{M_T}{K_a\gamma H^3} = \frac{\frac{1}{2}\alpha^3(1-\alpha)^3 m\rho(12\lambda' - \alpha^2) + \frac{\alpha}{6}[6K_{T_2}\lambda' - \alpha K p_2]}{\left[\frac{M_D}{M_T}(3\alpha K m_2 + (1-\alpha)K' m_2 + 6\alpha^2(1-\alpha)^3 m\rho) - (1-\alpha)K' m_2 + 6\alpha^2(1-\alpha)^3 m\rho\right]} \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

where $\lambda' = \dfrac{T}{K_a\gamma H^2}$ for the pinned pile.

Fig. 5



For an infinitely stiff pile, equation (15) gives :

$$\frac{M_T}{K_a\gamma H^3} = \frac{\alpha}{6}(3 - \alpha^2) \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Since an infinitely stiff pile encastré at the top cannot deflect at the toe, the pressure distribution is given by *Fig. 5*, whence, by taking moments about the anchorage, equation (16) is found to be correct.

The additional anchorage force arising from the encastré condition, extra to the pinned condition, is given by :

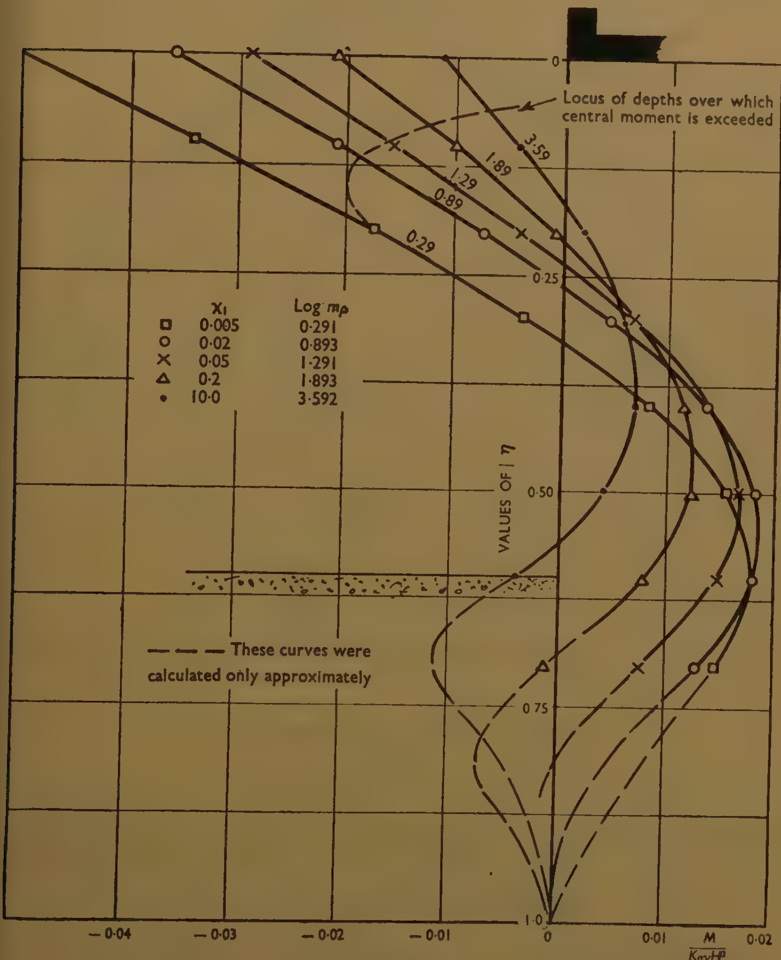
$$R = M_T \frac{\left(1 - \frac{M_D}{M_T}\right)}{\alpha H}$$

The additional coefficient is λ''.

$$\lambda'' = \frac{R}{K_a\gamma H^2} = \left(\frac{M_T}{K_a\gamma H^3}\right) \times \frac{1}{\alpha}\left(1 - \frac{M_D}{M_T}\right) \quad . \quad . \quad . \quad (17)$$

The total anchor load coefficient $= \lambda' + \lambda''$ . . . . (18)

Fig. 6

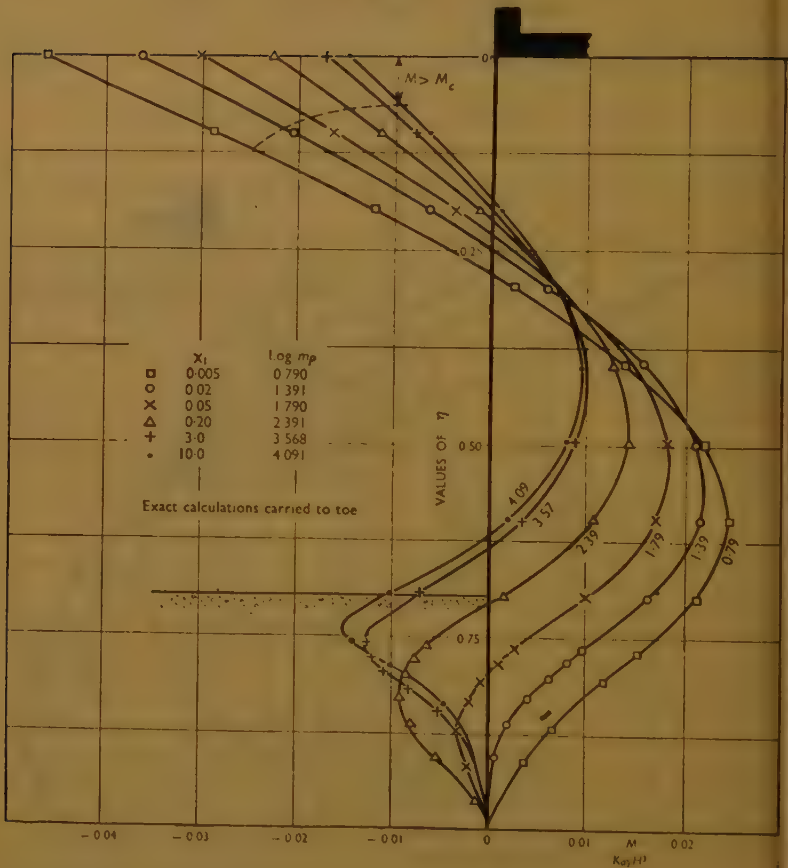


BENDING MOMENT DISTRIBUTIONS

$a = 0{\cdot}6$

(Units: M : lb/ft/ft; $K_a\gamma$: lb/cu. ft; H : ft)

Fig. 7



BENDING MOMENT DISTRIBUTIONS

$a = 0{\cdot}7$

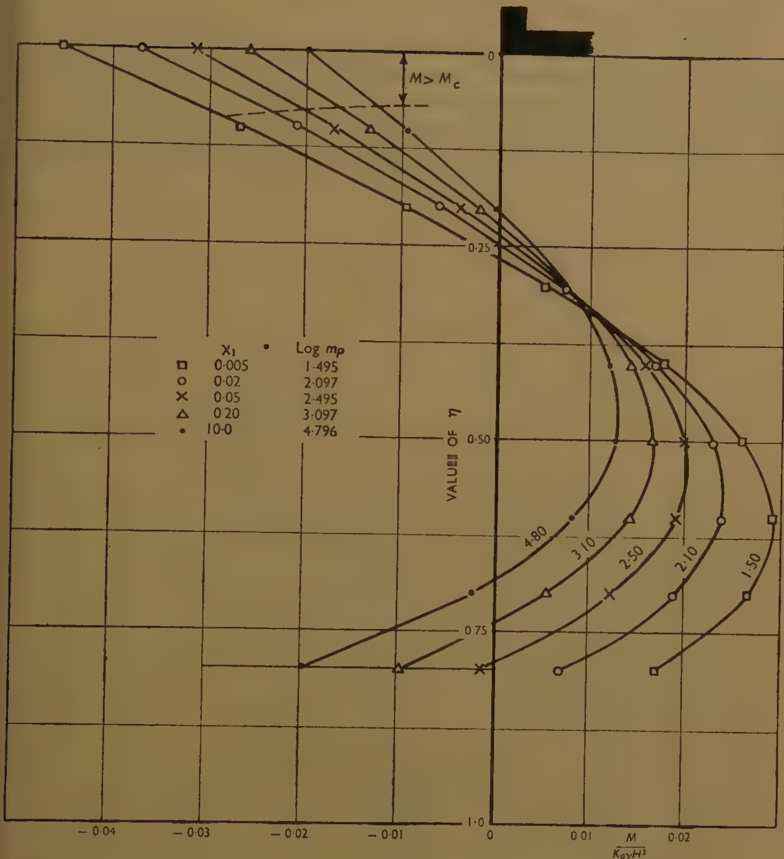
The bending moment at any point above the dredge level is now given by :

$$\frac{M_{(\eta H)}}{K_a\gamma H^3} = (\lambda' + \lambda'')\eta - \frac{\eta^3}{6} - \frac{M_T}{K_a\gamma H^3} \quad . \quad . \quad . \quad (19)$$

The largest numerical value of equation (19) is M_T, which is negative. The maximum positive value M_C above the dredge level is given by :

$$\frac{M_C}{K_a\gamma H^3} = 0{\cdot}9428(\lambda' + \lambda'')^{\frac{3}{2}} - \frac{M_T}{K_a\gamma H^3} \quad . \quad . \quad . \quad (20)$$

Fig. 8



[Bending moment distributions

$\alpha = 0{\cdot}8$

The distribution of bending moment below the dredge level arising from the moment, M_T, and thrust, R, alone, is given by:

$$\frac{M}{K_a\gamma H^3} = \frac{M_D}{K_a\gamma H^3}\left[-f_1 + \left(\frac{M_T}{M_D} - 1\right)\frac{(1-\alpha)}{\alpha}\,.\,\epsilon\,.\,f_2\right.$$

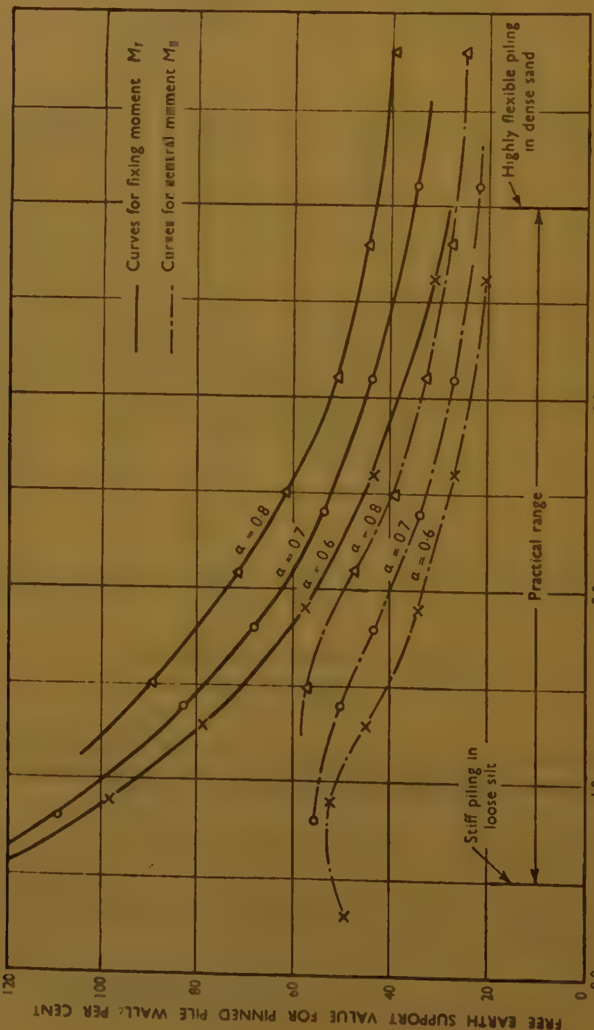
$$+\left\{4Km_1 - \left(\frac{M_T}{M_D} - 1\right)\frac{(1-\alpha)}{\alpha}K'm_1\right\}\epsilon^3 f_4 - \left\{3Km_2 - \left(\frac{M_T}{M_D} - 1\right)\right.$$

$$\left.\left.\frac{(1-\alpha)}{\alpha}\,K'm_2\right\}\epsilon^4 f_5\right] \quad . \quad . \quad (21)$$

where $f_1 - f_5$ have the values given in reference 2, p. 86. This distribu-tion must be added to that acting on the pinned pile—equations (11) and (18), reference 2, p. 86.

Values of the fixing moment, M_T (equation 15), and the distribution of moment above the dredge level (equation 19) have been calculated for a range of values of $\log_{10} mp$ for $\alpha = 0{\cdot}6$, $0{\cdot}7$, and $0{\cdot}8$; the resultant distributions below the dredge level have also been calculated for $\alpha = 0{\cdot}7$. The results are given in *Figs 6, 7,* and *8*. The calculations show that,

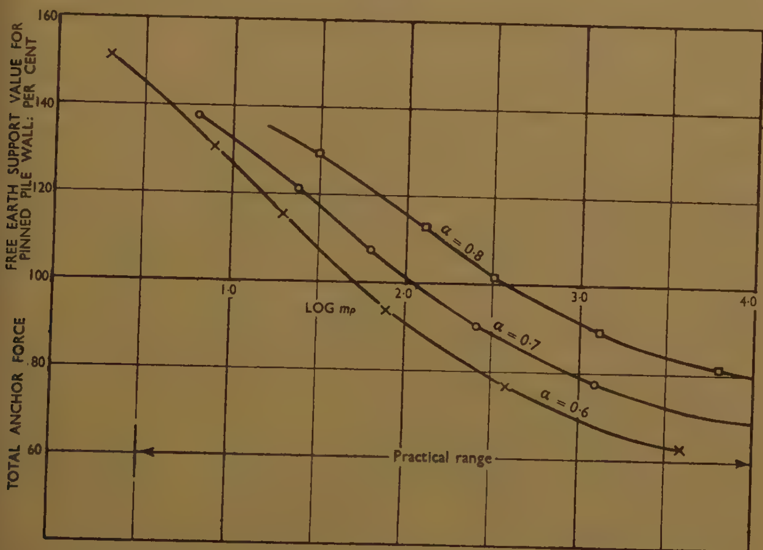
Fig. 9



REDUCTION CURVES FOR CENTRAL AND END FIXING MOMENTS

under normal working-stress conditions for practical ranges of pile flexibility and soil stiffness, the maximum bending moment beneath the dredge level is less than that above the dredge level. The fixing moment at the anchorage exceeds the central positive moment in value over a distance approximately $\frac{1}{10}H$ below the anchorage. The fixing moments, M_T, and maximum central moments, M_C, are plotted as a percentage of the maximum free earth support value for a pinned pile in *Fig. 9*. This diagram summarizes the essential design bending-moment data for the wall. In addition, the bending-moment distribution curves are of value in determining the distribution of steel reinforcement in concrete walls.

Fig. 10

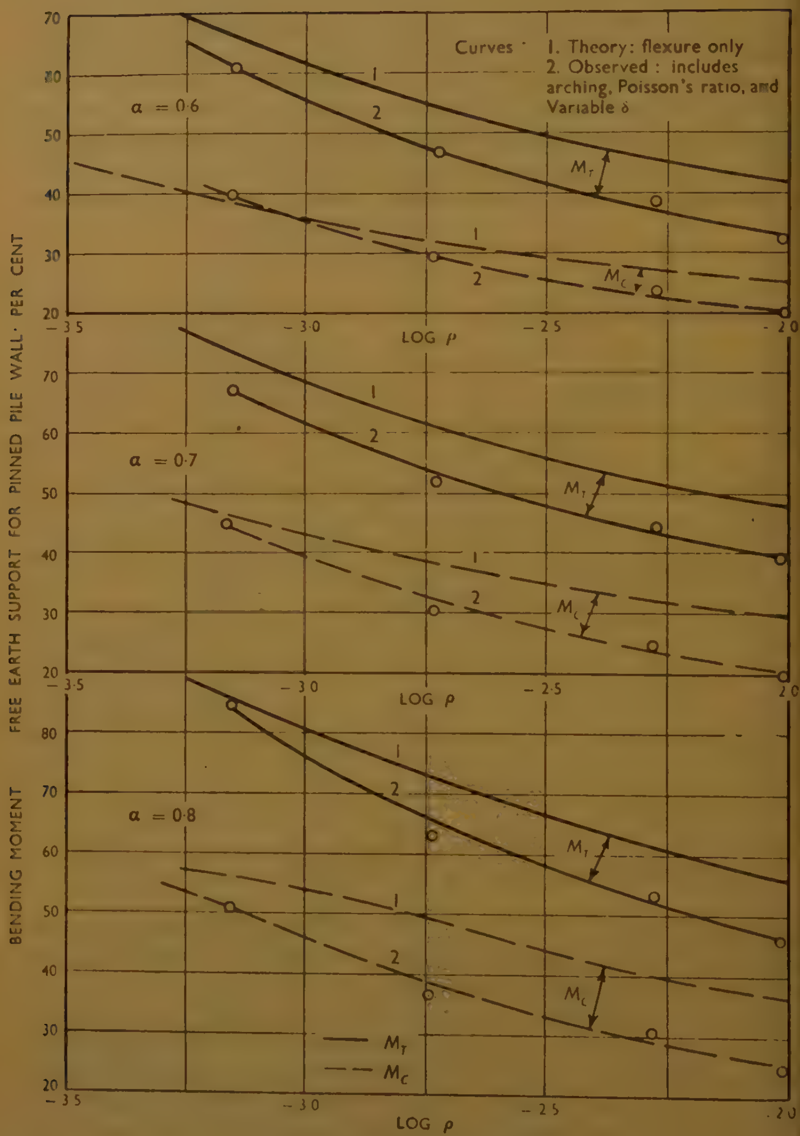


ANCHOR SHEAR FORCE VARIATION WITH FLEXIBILITY AND SOIL STIFFNESS

The maximum shear force occurs at the anchorage and the total anchor loads calculated from equation (18) are given as a percentage of the free-earth-support values for a pinned pile in *Fig. 10*.

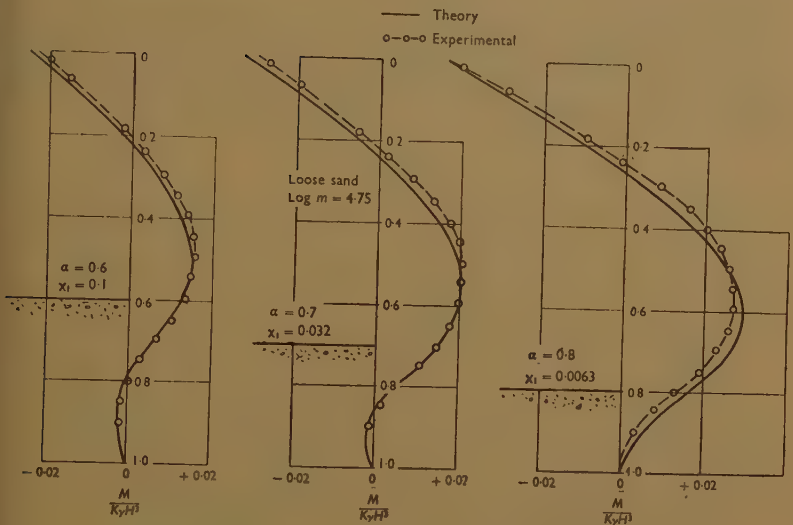
The above calculations were based on the temporary assumption of a triangular type of active pressure distribution above the dredge level. The results of tests on model sheet-pile walls 2 feet 6 inches high, encastré at the top, embedded in dry loose sand, are shown in *Figs 11* to *13*. For stiff walls, the bending-moment curves and maximum values agree closely with those calculated arising from flexure alone, *Figs 11* and *12*. With increase in flexibility, the observed moments fall below those from flexure alone. This further reduction was 20–30 per cent in the practical

Fig. 11



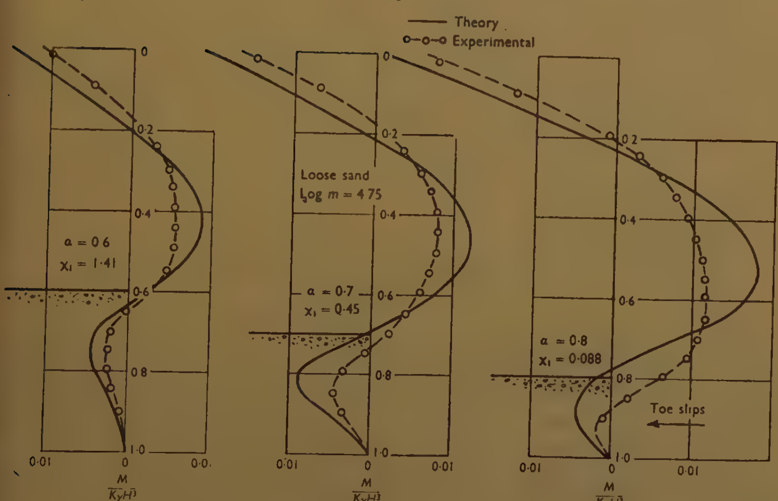
COMPARISONS WITH OBSERVATIONS

Fig. 12



BENDING MOMENT DISTRIBUTIONS. LOG $\rho = -3{\cdot}16$. LOOSE SAND

Fig. 13

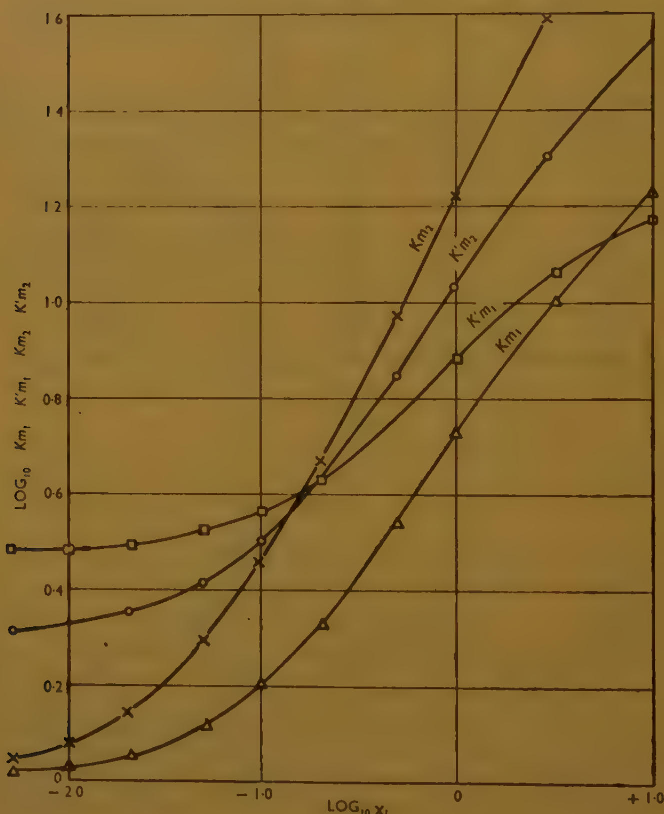


BENDING MOMENT DISTRIBUTIONS. LOG $\rho = -2{\cdot}01$. LOOSE SAND

flexibility range, arising from the influences of arching (20 per cent), and the Poisson's ratio effect (9 per cent).

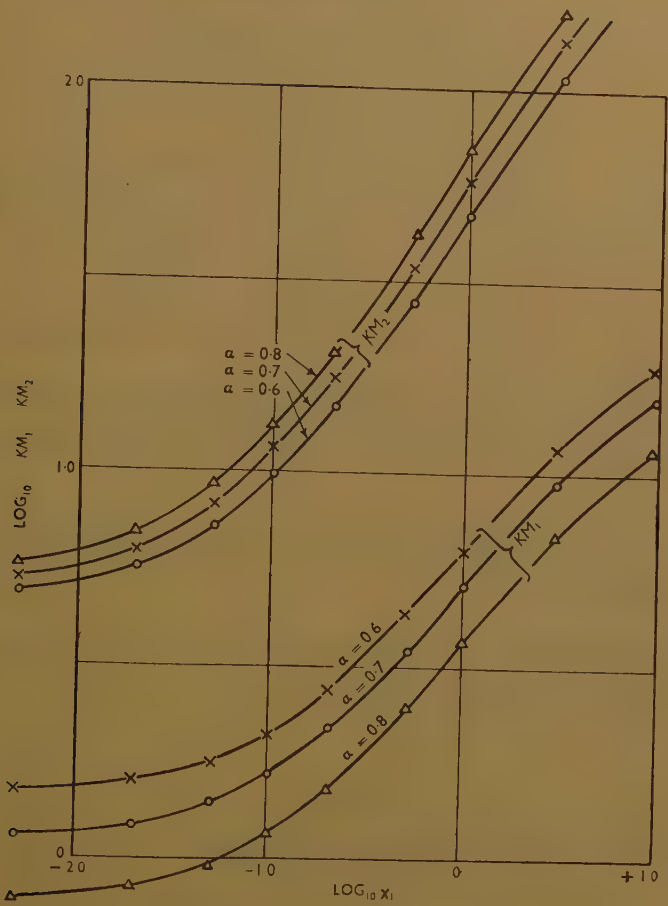
The bending-moment distribution curves for highly flexible piling, *Fig. 13*, show a progressive discrepancy between the theoretical and the observed points of zero bending moment below the dredge level, with dredging. This results from the influence of the passive outward slips

Fig. 14



of the pile toe which are equivalent to a reduction in the soil stiffness modulus, *m*. The increase of the value of M_C expected at low dredge levels ($\alpha = 0{\cdot}8$) is counteracted by a larger arching reduction factor because the arch spreads over the whole length of the pile. Further dredging leads to a rapid increase in the fixing moment towards the value on a wall cantilevered from the top and subject to arching over the full height.

Fig. 15



Summary of Design Procedure

(1) Estimate the soil stiffness modulus value from subsoil penetration tests.[2]

(2) Calculate the maximum bending moment and tie-rod load which would act on a pinned wall.

(3) Draw the moment/flexibility curves and shear force/flexibility curves from *Figs 9* and *10* using values from (1) and (2) above.

(4) Estimate the further reduction arising from the Poisson's ratio effect and arching by empirical data or calculation.[3]

(5) Combine the resultant moment/flexibility curve with structural curves [1] to obtain the design flexibility.

(6) Use *Figs 6*, *7*, *8*, and *9* to detail the reinforcement.

Acknowledgements

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The Author has benefited from discussions with Dr J. Brinch Hansen and Mr K. Mortensen.

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2. P. W. Rowe, "A Theoretical and Experimental Analysis of Sheet-Pile Walls." Proc. Instn Civ. Engrs., Pt I, vol. 4, p. 32 (Jan. 1955).
3. J. Brinch Hansen, "Earth Pressure Calculation." The Danish Technical Press, Copenhagen, 1953.

The Paper is accompanied by twelve sheets of drawings and three diagrams from which the Figures in the text have been prepared, and by the following Appendix.

CORRESPONDENCE on this Paper should be received at the Institution before the 15th May, 1955, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

Appendix

$$K m_1 = \frac{1 + 7{\cdot}50\chi_1 + 3{\cdot}596403573\chi_1^2 + 0{\cdot}356786067\chi_1^3 + 0{\cdot}011526283\chi_1^4 + 0{\cdot}000207100\chi_1^5}{\text{Divisor}}$$

$$K' m_1 = \frac{3 + 10{\cdot}666666667\chi_1 + 3{\cdot}532467542\chi_1^2 + 0{\cdot}272569532\chi_1^3 + 0{\cdot}007276387\chi_1^4 + 0{\cdot}000006919\chi_1^5}{\text{Divisor}}$$

$$K m_2 = \frac{1 + 20{\cdot}57142858\chi_1 + 14{\cdot}857142581\chi_1^2 + 1{\cdot}927753018\chi_1^3 + 0{\cdot}076095533\chi_1^4 + 0{\cdot}001256532\chi_1^5}{\text{Divisor}}$$

$$K' m_2 = \frac{2 + 15{\cdot}0\chi_1 + 7{\cdot}192807020\chi_1^2 + 0{\cdot}71357220\chi_1^3 + 0{\cdot}023052547\chi_1^4 + 0{\cdot}000231627\chi_1^5}{\text{Divisor}}$$

$$KM_1 = \frac{\begin{aligned}&(3 - 2\alpha - \alpha^2) + \chi_1[10{\cdot}6666 + 8{\cdot}6666\alpha - 19{\cdot}3333\alpha^2]\\ &+ \chi_1^2[3{\cdot}53246754 + 7{\cdot}3206789\alpha - 10{\cdot}8531465\alpha^2]\\ &+ \chi_1^3[0{\cdot}27256953 + 0{\cdot}8820054\alpha - 1{\cdot}15457487\alpha^2]\\ &+ \chi_1^4[0{\cdot}007276387 + 0{\cdot}03155232\alpha - 0{\cdot}038828707\alpha^2]\\ &+ \chi_1^5[0{\cdot}000006919 + 0{\cdot}000449416\alpha - 0{\cdot}000456335\alpha^2]\end{aligned}}{\text{Divisor}}$$

$$KM_2 = \frac{\begin{aligned}&(3 + 2\alpha + \alpha^2) + \chi_1[10{\cdot}6666 + 38{\cdot}6666\alpha + 74{\cdot}09523815\alpha^2]\\ &+ \chi_1^2[3{\cdot}53246754 + 21{\cdot}70629320\alpha + 63{\cdot}90409550\alpha^2]\\ &+ \chi_1^3[0{\cdot}27256953 + 2{\cdot}309149600\alpha + 8{\cdot}98479903\alpha^2]\\ &+ \chi_1^4[0{\cdot}007276387 + 0{\cdot}077657452\alpha + 0{\cdot}371639341\alpha^2]\\ &+ \chi_1^5[0{\cdot}000006919 + 0{\cdot}001277816\alpha + 0{\cdot}006254457\alpha^2]\end{aligned}}{\text{Divisor}}$$

$$\begin{aligned}\text{Divisor} = {} & 1 + 1{\cdot}142857130\chi_1 + 0{\cdot}197802211\chi_1^2 + 0{\cdot}009642718\chi_1^3\\ & + 0{\cdot}000180303\chi_1^4 + 0{\cdot}000000247\chi_1^5\end{aligned}$$

Paper No. 5998

"The Computation of Shrinkage and Thermal Stresses in Massive Structures"

by

* Olgierd Cecil Zienkiewicz, Ph.D., B.Sc.(Eng.), A.M.I.C.E.

(Ordered by the Council to be published with written discussion)

SYNOPSIS

The Paper presents an extension of relaxational techniques for dealing with some problems of shrinkage and thermal stress encountered in massive concrete structures.

Although it is realized that no mathematical solution can fully present an answer to such complex cases as occur in practice, an approximation at least to the true stress distribution may be achieved. Several examples of shrinkage problems frequently encountered in practice are worked out and the relatively complex distribution of stresses is shown. The problems worked out include stresses in heightening of a gravity dam, and the effects of shrinkage of concrete masses placed in contact with rock foundations.

INTRODUCTION

DESPITE the well-known fact that the shrinkage and thermal effects manifested in massive concrete structures may cause stresses in excess of those resulting from the loads the structure may be carrying, their analysis has received very little attention compared with similar effects on simpler structures such as arches, etc. The reasons for this are many : the thermal conditions are difficult to assess ; the amounts of shrinkage may vary throughout the body of the structure on account of its building-up process ; the complex stress/strain relations, etc. All these create a type of problem, to which an exact solution may never be found. Even with numerous simplifying assumptions, the problem, in all but the very simplest cases, defies general mathematical solutions. It is the object of this Paper to show that, with some basic assumptions, it is possible to obtain results even for quite complex cases by the use of relaxational techniques. Although these results may still be only a crude approximation to the actual stresses developed, they should be valuable in assessing thermal shrinkage effects in important structures.

* The Author is Lecturer, Sanderson Engineering Laboratories, Edinburgh University.

Some attempts at finding solutions to similarly idealized problems as those described in the Paper have been made, using photoelastic techniques, by Smits[1] and others. Although reasonably consistent answers can be obtained by such methods, they seem only practicable in the simplest cases and lack the versatility of relaxation methods for more complex conditions.

Basic Assumptions of the Treatment

To allow the problem to be treated exactly, it is necessary to make the following assumptions :—

(1) The material is homogeneous and isotropic with constant elastic properties.
(2) Known temperature and shrinkage values are applied to the structure *after* the material has set and has obtained the above properties.
(3) Solutions are limited to two-dimensional cases of plane stress or plane strain.

It will be evident to those with experience of concrete—which is generally the material involved—how much these hypothetical conditions are departed from in practice. Assumptions of constant elastic properties are obvious approximations, and the process of shrinkage and temperature development, occurring simultaneously with the hardening, causes a further discrepancy.

The limitation to two-dimensional cases curtails the applicability of the solution to some extent, but it does give a good approximation in the vicinity of central sections of three-dimensional structures.

Values of temperature distribution in massive structures can be obtained by methods similar to those described by Professor Ross,[2] and frequently, in cases of such structures as gravity dams, by actual measurement in situ.

Determination of shrinkage values has to be limited necessarily to laboratory specimens. Shrinkage coefficients for concrete range from $\times 10^{-4}$ to 7×10^{-4} according to various sources.

General Equations

Because temperature and shrinkage stresses are basically of the same type—in both cases caused by the tendency of the material to change its volume and by the restraints of the boundaries or adjacent parts of the ructure—the latter can be treated as a specific case of the former.

The necessary equations which require solution are best provided by e use of Airy's stress function. Referring to the stress system given in

[1] The references are given on p. 99.

Fig. 1, it can be shown easily that, if the stresses are defined by the use of the Airy stress function ϕ as follows :

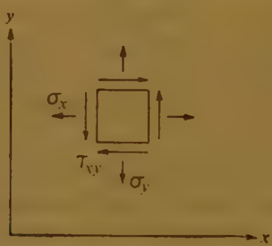
$$\sigma_x = \frac{\partial^2\phi}{\partial y^2}$$

$$\sigma_y = \frac{\partial^2\phi}{\partial_x^2}$$

$$\tau_{xy} = \frac{\partial^2\phi}{\partial_x\partial_y} \quad \text{. . . . .} \quad (1)$$

then the conditions of equilibrium of the element are automatically satisfied, providing that no body forces are acting.

Fig. 1



DEFINITION OF STRESSES

Consideration of strains caused by the above stresses and temperature changes leads to the following equations which have to be satisfied in the case of plane strain by the function ϕ :

$$\nabla^4\phi = -\frac{E}{1-\nu}\nabla^2(\alpha T) \quad \text{. . . . . .} \quad (2a)^*$$

or in the case of plane stress :

$$\nabla^4\phi = -E\nabla^2(\alpha T) \quad \text{. . . . . .} \quad (2b)^*$$

In the foregoing equations the following notation is used :—

E denotes Young's modulus
ν ,, Poisson's ratio
α ,, coefficient of thermal expansion
T ,, temperature

$$\nabla^2(\ ) \equiv \frac{\partial^2(\ )}{\partial x^2} + \frac{\partial^2(\ )}{\partial y^2}$$

* Deduction of equations (1) and (2) can be checked easily from the standard equations of compatibility and equilibrium. The Author notes that a form of equation similar to, but less convenient than, that of equations (2) is quoted on p. 424 of reference 3.

$$\nabla^4(\ )\equiv\frac{\partial^4(\ )}{\partial x^4}+2\frac{\partial^4(\ )}{\partial x^2\partial y^2}+\frac{\partial^4(\ )}{\partial y^4}$$

The solution of the problem of stress distribution, arising from a known temperature state, resolves itself thus to finding a function ϕ which satisfies equations (2) at all points at the same time, giving the required values of stresses on the external boundaries. Since the stresses arising from external loads are generally known, the temperature stresses can be treated separately. With no boundary loading, it is then evident that the required free boundary conditions will be :

$$\frac{\partial\phi}{\partial x}=\mathrm{A}\,;\ \frac{\partial\phi}{\partial y}=\mathrm{B}\,;\ \text{and}\ \ \phi=\mathrm{A}x+\mathrm{B}y+\mathrm{C}$$

Because the constants, A, B, and C can take on any arbitrary value without affecting the stresses in most of the problems solved here, they were made equal to zero.

It should be noted that a solution obtained for a plane strain case may be simply translated into that of plane stress by a change of a coefficient.

Since αT represents the proportional increase in length of an element resulting from a temperature rise, T, it should at this stage be observed that, in problems where shrinkage is involved, substitution of $\alpha T=-s$ —value of shrinkage at point considered—enables shrinkage problems to be dealt with on identical lines.

Before attempting a solution of equations (2), it is convenient to present the equation in a non-dimensional form, thus simplifying, to some extent, the numerical work involved. Taking, in the case of plane stress :

$$x=Lx'$$
$$y=Ly'$$
$$T=\theta T_0$$
$$\phi=\frac{EL^2\alpha T}{1-\nu}\psi$$

L and T_0 representing some arbitrary length and temperature, the governing equation (2) reduces to

$$\nabla^4\psi+\nabla^2\theta=0 \qquad \ldots\ldots\ldots \qquad (3)$$

in which the differentiations are carried out with respect to the dimensionless co-ordinates x' and y'.

Solution of the Equations

Full descriptions of relaxational techniques, as applied to similar equations, appear elsewhere [4, 5], so only a summary of the procedure is given in this Paper.

Step I A square mesh is drawn throughout the region considered, and the governing and boundary equations put in the finite difference approximation using only the values of the functions of the mesh points. Referring to *Fig. 2*, for example, the governing equation (3) applicable at point O, can be written as follows :

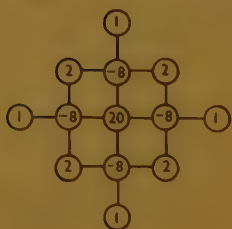
$$20\psi_0 + 2(\psi_5 + \psi_6 + \psi_7 + \psi_8) + \psi_9 + \psi_{10} + \psi_{11} + \psi_{12} - 8(\psi_1 + \psi_2 + \psi_3 + \psi_4) + a^2(\theta_1 + \theta_2 + \theta_3 + \theta_4 - 4\theta_0) = 0$$

Similar equations can be written down at all mesh points and, supplemented by the boundary conditions, give a system of simultaneous

Fig. 2

Relaxation mesh

Fig. 3



Relaxation pattern

equations equal in number to the mesh points considered. With known values of θ, the solution of this system gives the required values of ψ at all mesh points.

Step II To solve this large number of simultaneous equations, relaxation technique is followed. Initial values of ψ are guessed at all mesh points and "residuals," representing the values of the left-hand side of equation (4), are calculated. Then, by successive corrections of the individual ψ values, the residuals are reduced until they become insignificant. *Fig. 3* represents what is known as a "relaxation

pattern" applicable to equation (3), showing the change in residuals at points surrounding O when ψ_0 is changed by unity.

Step III The same procedure is repeated on a finer mesh to obtain a more accurate solution. Initial values here are obtained by interpolation from the first solution, reducing the amount of "relaxation"

***Fig. 4** (a)*

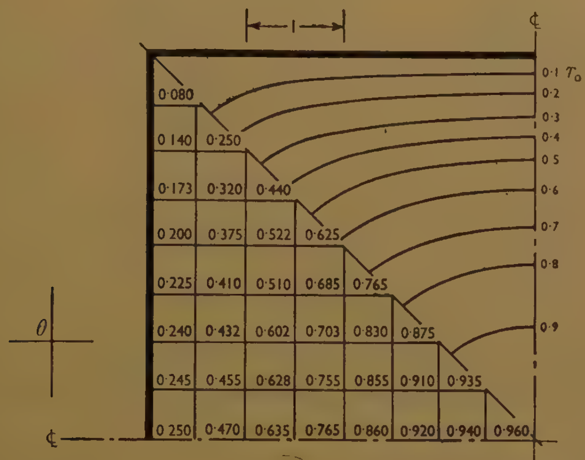
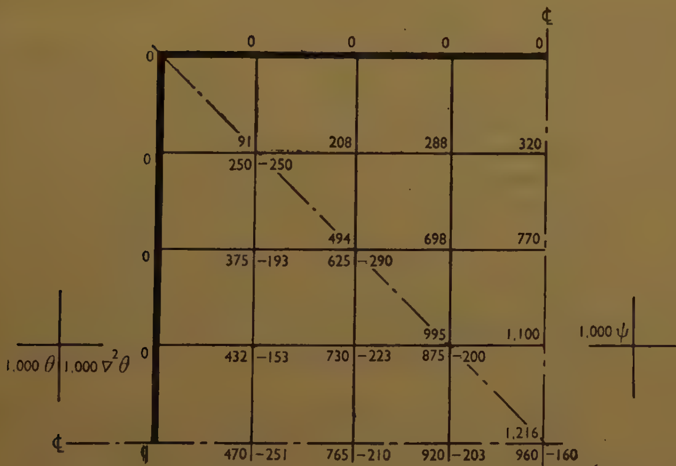
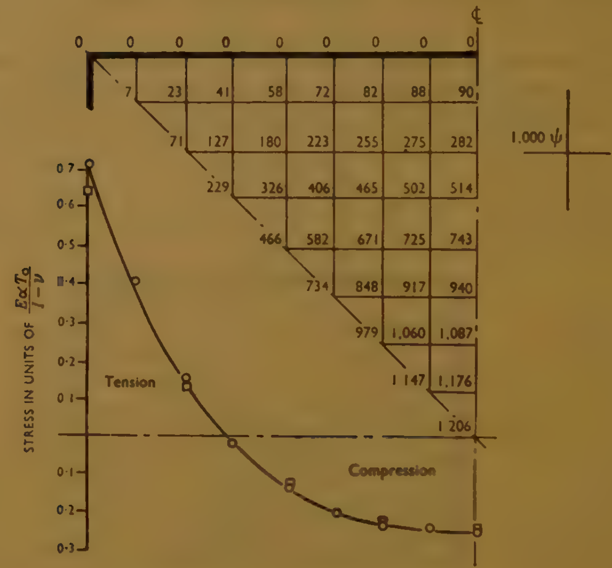


***Fig. 4** (b)*



STRESSES DURING COOLING OF A SQUARE PRISM (see also *Fig. 4* (*c*))

Fig. 4 (c)



STRESSES DURING COOLING UP A SQUARE PRISM (see also *Figs 4 (a)* and *(b)*)

required. If the changes in ψ are not significant at this stage, the solution has been obtained within the required accuracy.

Step IV Stresses at all points can be computed by using the appropriate finite difference approximations to equation (1).

A simple example is shown in *Figs 4 (a)*, *(b)*, and *(c)*. It represents the solution to the thermal stresses produced at a stage of cooling of a long square-section prism which was initially at a temperature T_0 and is being cooled by exposing the outer boundaries to a temperature of zero. The temperature distribution assumed at the particular instant is given in *Fig. 4 (a)* and is interpolated from the isotherms given by Carslaw and Jaeger.[6] *Fig. 4 (b)* shows the solution obtained on a fairly coarse mesh, and *Fig. 4 (c)* a better approximation on a mesh of half the size. On the same diagram is plotted the distribution of normal stresses on a centre-line section. The values obtained from the coarse-mesh solution are very close to those of the more accurate solution and the general level of accuracy can be easily ascertained.

The distribution of stresses is interesting, showing the expected tensile stresses near the outer boundary, and the accompanying compression within the central core.

This relatively simple example could be solved, with considerable

labour, by means of an orthodox analytical approach. It should be observed, however, that the relaxation technique can be extended quite simply to any shape of a boundary, and a practical problem of, say, the distribution of stresses in a gravity dam caused by known temperatures would not present any difficulties.

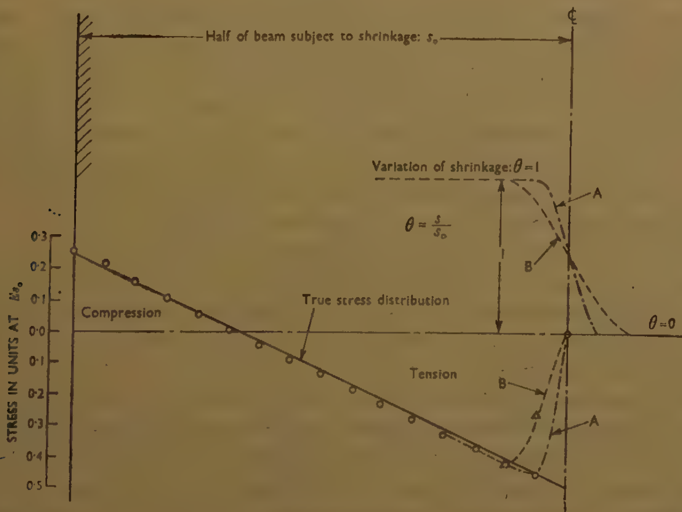
Discontinuous Shrinkage

Although problems with shrinkage varying in a continuous way across the body of the structure could be tackled in an identical way, most problems of shrinkage in practice are those where a part of the structure is subject to uniform shrinkage and is attached to a part which does not shrink. Such cases arise, for example, when fresh concrete is bonded to old, or to a foundation of rock, and are characterized by a discontinuity of s, the shrinkage, at this boundary.

To treat such cases by the methods described earlier, it is necessary to assume that the shrinkage varies continuously within a narrow region adjacent to the interface. Physically, such an assumption, by the principle of Saint Venant, will effect the stresses only in the immediate neighbourhood of the discontinuity. Mathematically, if the region of variation of s is small, say, limited to one mesh-length either side of the interface, errors in the finite difference approximations may be expected.

To test the method and illustrate the points in question a simple example will be given. This is a case of a long rectangular beam, half of which,

Fig. 5



Stresses in a beam half of which is subject to shrinkage

situated below the neutral axis, is subject to uniform shrinkage, s_0. Such a problem being identical to that of a long bi-metallic strip subject to a temperature change, can be treated by methods outlined in most standard texts on strength of materials and the known solution compared with that obtained relaxationally. *Fig. 5* shows the solution. The details of working follow those described previously; the equations now are rather simplified since ψ and $\theta \left(= \frac{s}{s_0}\right)$ do not vary in the direction of the beam axis. The regions of variation of s of two and four mesh-lengths were both tried, to ascertain the effects of the poor finite-difference approximations there. Comparison of the resulting curves of stress distribution with the standard solution shows that :—

(1) The stresses in the major part of the beam are evaluated correctly by the method.

(2) The error arising from assuming the variation of s to occur within two mesh-lengths is small.

(3) The stresses at the interface, being finite, can be obtained with good accuracy by extrapolating the main stress distribution curves.

The methods outlined can now be applied to some problems of practical interest to the civil engineer.

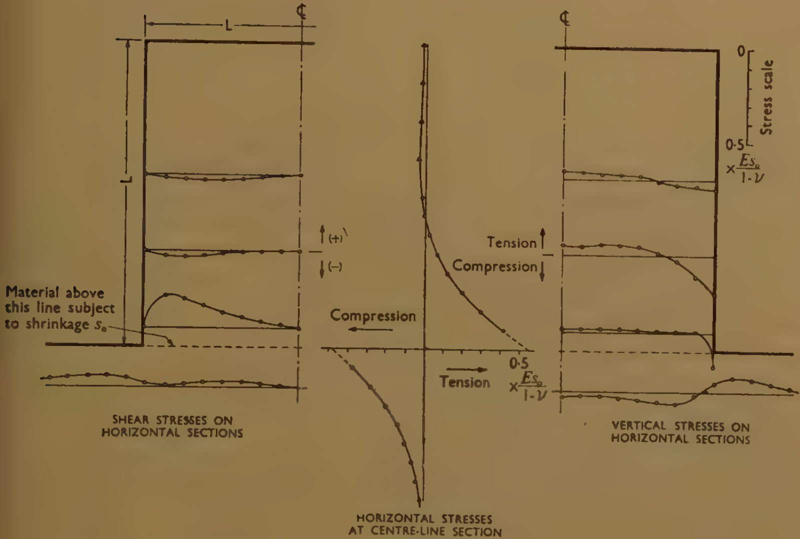
Mass of Concrete in Contact with a Rock Foundation

This is a frequently encountered problem and it is well known that, on occasion, stresses of magnitudes capable of initiating cracks can be developed. It is assumed, for the purposes of the example, that the modulus of elasticity of the foundation is the same as that of the concrete and that the latter is subject to a uniform shrinkage s_0. *Fig. 6* shows the distribution of stresses for the case of a mass of square section. It should be noted that, because the extent of the foundation is infinite, the "boundary" conditions within its body are given by the tendency of the stresses to become zero and hence a constant value of ψ.

Although, strictly speaking, the solution is valid for a block of infinite length, it is reasonable to assume that, for a central section of a block of finite length, the solution will be identical especially since Poisson's ratio for concrete has very small value. *Fig. 7* illustrates the stresses obtained with a rectangular block.

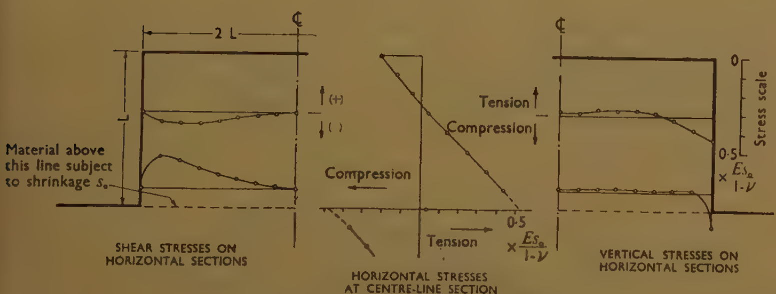
The distribution of normal stresses on the vertical section is of interest. Near the base, tensile stresses approaching 0·6 of the fully restrained values are encountered but tend to decrease rapidly with the height. Reversal of the direction of these stresses at some height above the base is perhaps, unexpected. The experiment carried out by Smits[1] on triangular sections rather tends to bear out this point.

Fig. 6



Sqare Block subject to Shrinkage on an Elastic Foundation

Fig 7.



Rectangular block subject to shrinkage on an elastic eoundation

It is obvious from *Fig. 6* that, with blocks where the height is greater than the base width, stresses tend to zero in the upper portions. At the other end of the scale, with blocks of very great width, stresses will tend to become uniform over a vertical section and equal in value to full restraint.

It is of some interest in this connexion that the Kammüller formula for the spacing of contraction joints in dams, quoted by Leliavsky,[7] assumes a

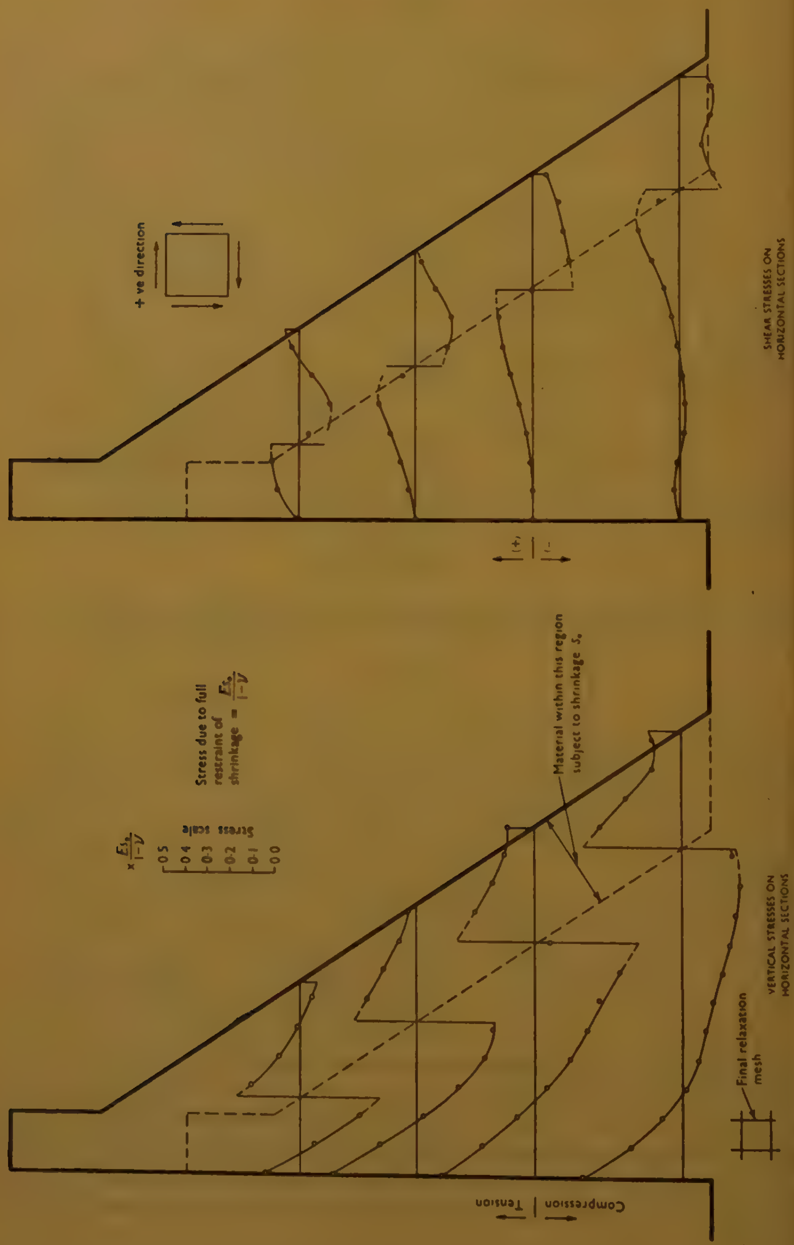

+ ve direction
SHEAR STRESSES ON HORIZONTAL SECTIONS
Stress due to full restraint of shrinkage = $\frac{Es_0}{1-\nu}$
$\times \frac{Es_0}{1-\nu}$
0·5 0·4 0·3 0·2 0·1 0·0
Stress scale
Material within this region subject to shrinkage s_0
VERTICAL STRESSES ON HORIZONTAL SECTIONS
Final relaxation mesh
Compression
Tension

Fig. 8

uniform distribution of stress and thus to some extent underestimates the opening of the joints resulting from shrinkage.

Enlargement of an Existing Dam

In this case, appreciable stresses can be caused by the shrinkage of the new layer of concrete bonded to the old material. That relatively large stresses can be caused by so doing is realized by engineers acquainted with the problem and is illustrated by elaborate precautions which had to be taken in enlarging the Mullardoch dam.[8] *Fig. 8* shows the distribution of some of the stresses caused by a uniform shrinkage of the outer layer in a typical example. Since these stresses have to be superposed on those caused by the weight of the structure and water pressure, it can be seen that large tensile stresses may be caused not only in the new layer, where cracking could possibly be tolerated, but also in the dangerous region of the upstream face. Taking the shrinkage value s_0 as $3{\cdot}5 \times 10^{-4}$, the tensile stresses at the upstream face may reach 300 lb. per square inch in this case assuming $E = 2 \times 10^6$ lb. per square inch and $\nu = 0$.

It should be noted that, in all the foregoing examples, the stresses produced are dependent only on the geometry of the structure and the shrinkage or temperature values; the size is of no consequence.

Conclusions

It is hoped that the method of computation of shrinkage and thermal stresses may be useful in structures of unusual type and importance and that the examples worked out will, by illustrating the type of stress distribution encountered, contribute to a better knowledge of the phenomena involved.

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The Paper is accompanied by three sheets of diagrams, from which the Figures in the text have been prepared.

CORRESPONDENCE on this Paper should be received at the Institution before the 15th May, 1955, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

CORRESPONDENCE
on a Paper published in Proceedings, Part I, May 1954

Paper No. 5957

"Photoelastic Experiments on the Stress Distribution in a Diamond-Head Buttress Dam"†

by

Professor A. W. Hendry, Ph.D., B.Sc., A.M.I.C.E.

Correspondence

Mr S. P. Christodoulides observed that the Author's investigation had apparently been carried out in 1950, with the result that the Paper underestimated the possibilities of the "frozen stress" technique. Work done since 1950, both practical and theoretical, confirmed that, contrary to the Author's statement on p. 371, it was possible to obtain the three principal stresses, P, Q, and R, and their directions, with reasonable accuracy and relative ease.

"Araldite B"[7] hot-setting resin had been used for several years for "frozen stress" models, and had proved itself a promising photoelastic material, superior to "Marco" and "Catalin." It lent itself to easy machining and, provided the models were kept in a desiccator, the initial stresses were small.

The fringe value of hot Araldite was 1·39 lb. per square inch/fringe/inch* at 130° C., which compared favourably with 1·6 at 85° C., for "Catalin 800", at 85° C.[8] The elastic moduli of Araldite were $4{\cdot}2 \times 10^5$ and 1,950 lb. per square inch at room and "freezing" temperatures respectively. The corresponding figures for "Catalin 800" were $1{\cdot}87 \times 10^5$ and 1,500 lb. per square inch.

The directions of the principal stresses, and the principal-stress differences, $P-Q$, $Q-R$, and $R-P$, could be obtained from observations in a "tilting stage,"[9] on slices taken from a "frozen" model. If the values of the stresses were required at points along a line, say Oy, slices could be

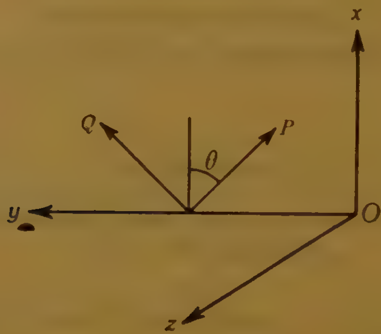
† Proc. Instn Civ. Engrs, Part I, vol. 3, p. 370 (May 1954).

* For definition of units of fringe value see p. 375 of reference †.

[7] References 7 to 11 are given on p. 106.

cut, from two identical models, in perpendicular planes containing the line Oy (see *Fig. 25*).

Fig 25



Calling these the xy and yz planes, and using two-dimensional photo-elastic methods, the shear stresses $\widehat{xy}$ and yz could be measured and the quantities $\frac{\partial \widehat{xy}}{\partial x}$ and $\frac{\partial \widehat{yz}}{\partial z}$ could be calculated.

Then, applying the equation of equilibrium in Cartesian co-ordinates :

$$\frac{\partial \widehat{xy}}{\partial x} + \frac{\partial \widehat{yy}}{\partial y} + \frac{\partial \widehat{yz}}{\partial z} = 0$$

integration could be carried out along Oy, to obtain $\widehat{yy}$, starting from a point where $\widehat{yy}$ was known. At a point on an unloaded boundary $\widehat{yy} = 0$.

If l, m, n denoted the direction cosines of Oy with respect to the principal directions, as established from the tilting-stage observations, it was known that :

$$\widehat{yy} = (P-R)l^2 + (Q-R)m^2 + R$$

in which all quantities except R were known, and therefore R could be evaluated. Having found R, the other two principal stresses P and Q were computed from the stress differences $P-R$ and $Q-R$ obtained from the tilting-stage measurements.

Alternatively, the observations on the tilting stage could be dispensed with, and three integrations carried out to obtain $\widehat{xx}$, $\widehat{yy}$, and zz ; then, from the equations :

$$\begin{cases} P + Q + R = \widehat{xx} + \widehat{yy} + \widehat{zz} \\ PQ + QR + RP = \widehat{xx} \times \widehat{yy} + \widehat{yy} \times \widehat{zz} + \widehat{zz} \times \widehat{xx} \\ PQR = \widehat{xx} \times \widehat{yy} \times \widehat{zz} ; \end{cases}$$

P, Q, and R could be calculated by solving the simultaneous equations, which would lead to a cubic. The method was rather tedious and not very accurate.

In the case of a plane of symmetry the work involved in arriving at values of the principal stresses was considerably less. If xy was a plane of symmetry, it was necessary to integrate only once to obtain yy. The well-known relation :

$$\widehat{yy} = \tfrac{1}{2}(P + Q) + \tfrac{1}{2}(P - Q)\cos 2\theta$$

where θ denoted the angle of the P–principal direction with the x–axis, could be used to obtain $P + Q$, since $P - Q$, and θ were obtainable from direct photoelastic observations on a slice cut to contain the xy–plane.

The values of P and Q could thus be calculated from their difference and sum ; by combining those values with direct measurements on a slice cut to contain the xz–plane, the third stress could be determined.[10, 11]

The models used in the experiments for the diamond-head buttress dam had a plane of symmetry, and the slices shown on *Figs 10*, p. 384, would be suitable for the application of the method outlined above.

In simulating gravitational forces by centrifuging it was essential to remember that the equations of elasticity satisfied in the two cases were not identical.

Thus, Beltrami's equation was :

$$\nabla^2(P + Q + R) + \frac{1-\eta}{1+\eta}\,\rho\overrightarrow{\nabla}(\overrightarrow{F} - \overrightarrow{f}) = 0$$

where the operators ∇^2 and $\overrightarrow{\nabla}$ were $\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial x^2}$ and $\hat{\imath}\frac{\partial}{\partial x} + \hat{\jmath}\frac{\partial}{\partial y} + \hat{k}\frac{\partial}{\partial z}$ respectively, $\hat{\imath}$, $\hat{\jmath}$, $\hat{k}$ being the unit vectors ; η denoted Poisson's ratio, $\overrightarrow{F}$ and $\overrightarrow{f}$ the gravitational and acceleration vectors respectively, and ρ the density.

When gravitational forces only existed, $\overrightarrow{F} = gk$, $\overrightarrow{f} = 0$, (g denoting the acceleration due to gravity), and $\overrightarrow{\nabla}\overrightarrow{F} = 0$.

Therefore Beltrami's equation became $\nabla^2(P + Q + R) = 0$, and the stress sum satisfied Laplace's equation.

When acceleration forces only existed, $\overrightarrow{F} = 0$, $\overrightarrow{f} = \hat{r}w^2r$ ($\hat{r}$ denoting the unit vector, r the radius, and w the angular velocity), and $\overrightarrow{\nabla}\overrightarrow{f} = 3w^2$; then Beltrami's equation became :

$$\nabla^2(P + Q + R) + 3w^2\rho(1 - \eta)/(1 + \eta) = 0$$

The two different equations indicated different states of stress under gravitational and centrifugal forces. Also, allowance for the additional term would clearly have to be made if relaxation methods were to be

used for "splitting" the stresses. Furthermore, where such methods were used on three-dimensional slices, assumed as two-dimensional, the third stress R involved in both the above equations was ignored and the three-dimensional operator ∇^2 was treated as a two-dimensional one. Those approximations introduced errors whose magnitude could well be of importance. The disagreement between the experimental and theoretical results shown on *Figs 8* and *11*, and stated on p. 382, were of a serious nature. One should not expect discrepancies much worse than 10 per cent in photoelastic work, except where only a qualitative analysis was intended.

In the theoretical investigation, according to assumption No. 2, p. 372, "The stress through the thickness of the material at any distance x from the upstream face is uniform." That was not compatible with *Fig. 12* and *Fig. 16* on which a very pronounced variation of the stress differences was photographed. A state of uniform stress would produce uniform principal stresses and principal-stress differences.

The Author, in reply, said that before dealing with the various points raised by Mr Christodoulides it was perhaps worth reiterating that the purpose of the investigation had been to establish that the design method of calculation was sufficiently accurate and to examine the stress distribution in the diamond head of the dam. As pointed out in the Paper there could be no question of extreme accuracy in the stress analysis of a concrete dam and, therefore, in the present investigation any elaboration of the method of analysis which in the end led to corrections of only a few per cent would have been futile. Again, since the work was for practical design purposes, the time available was limited and use had to be made of materials and techniques available when the experiments were put in hand. Mr Christodoulides's reference to "Araldite," although interesting, was therefore hardly relevant.

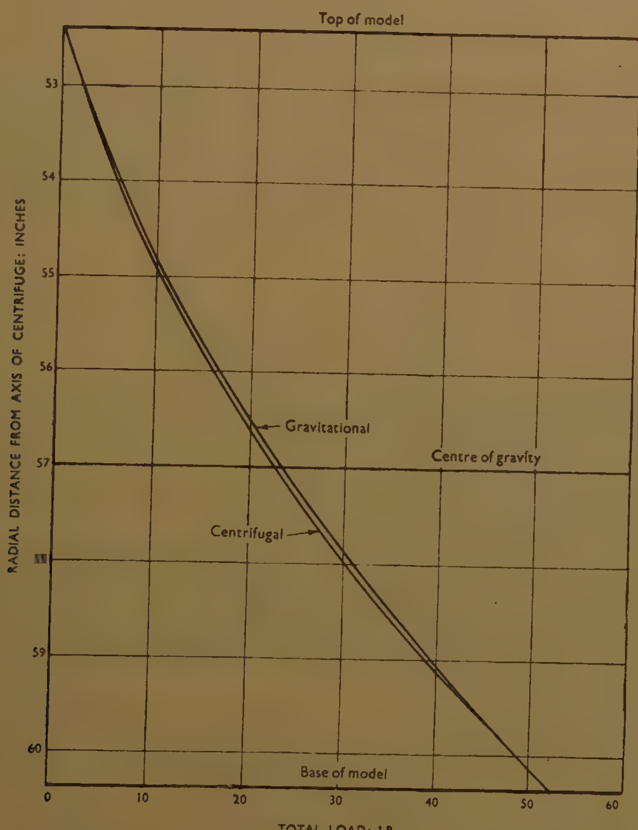
Although the methods described by Mr Christodoulides for determining the separate principal stresses could no doubt have been applied to the analysis of the diamond head of the model it was very doubtful whether any information would have emerged commensurate with the trouble and expense which that would have entailed. At least one more model would have been required and the cost of making and slicing models was sufficiently high to restrict the number used to the essential minimum. The use of three-dimensional techniques had been considered but those were not in fact applied when it was confirmed by comparison with an actual two-dimensional model that the stress system in a slice of the diamond head normal to the water face could be considered as two-dimensional (see *Fig. 17*). Naturally, agreement was not perfect but it was sufficiently good for the purposes of the tests.

The Author agreed that considerable advances in three-dimensional photoelasticity had taken place during the past few years but he would suggest that the method was at about the same stage of development as

two-dimensional techniques had been 20 years ago, when materials were still imperfect and computation of the principal stresses was effected by a step-by-step method of rather doubtful accuracy.

The difference between stresses set up by gravitational and centrifugal acceleration fields in relation to the buttress dam experiments was quite

Fig 26



TOTAL VERTICAL LOAD ON HORIZONTAL SECTIONS OF MODEL

easily explained and assessed without resort to the theory of elasticity. In a gravitational field vertical acceleration was uniform over the height of the model whereas in a centrifugal field the acceleration varied with the radius from the centre of rotation and in the model was, therefore, greater at the base than at the top. It was obvious, however, that if the height of the model was small compared to the radius the acceleration would be

practically uniform over the whole height and a gravitational field closely approximated. The extent to which that had been achieved in the experiment might be judged from *Fig. 26* which showed the total vertical load acting on horizontal sections of the model in a true gravitational field and in the centrifugal field actually used. The difference was negligible.

Mr Christodoulides's remarks concerning errors were not easily understood. The shear stresses shown in *Figs 8* and *11* did not involve the approximations of which he complained, and in any case the design method of calculation could not form a basis for the estimation of error since it was in itself liable to be even more seriously in error than the photoelastic work. The fact that the stress through the thickness of the material was not uniform was pointed out in the Paper (p. 383). Had the investigation been intended as an academic exercise then, of course, the various effects mentioned by Mr Christodoulides would have required closer attention; the object, however, had been to provide information of significance to the designer and in that respect the Author was encouraged to believe that the work was successful.

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ELECTION OF MEMBERS AND ASSOCIATE MEMBERS

The Council, at their meetings on the 19th October, and the 16th November, 1954, in accordance with By-law 14, declared that the under-mentioned had been duly elected.

Members

Archer, George Charles.
Bierrum, Hans Adolf, B.Sc. (*Copenhagen*).
Ings, Jasper Harold, B.A.Sc.(*Toronto*).
Leishman, Oswald Thomas Russell.
Parry, Cyril, *D.F.C.*
Plumpton, Mark William.
Wakeford, John Chrysostom Barnabas, *C.M.G.*
Williams, John Norman.

Associate Members

Anderson, Eric, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Barnett, Norman Nathaniel, Grad. I.C.E.
Birkett, Caryll Wreford, B.A. (*Oxon*), Grad.I.C.E.
Blower, Donald William, B.Sc. (*Glasgow*).
Brien, Donald, B.Sc.Tech. (*Manchester*).
Carey, Harry Neill, Ph.D., B.Sc. (*Durham*), Grad.I.C.E.
Coon, Robert James, Grad.I.C.E.
Cunningham, Peter Ian Frederick, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Darlow, Robert Brockway, B.E. (*New Zealand*).
Dearlove, Dennis, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Direktor, Leo, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Douglas, Thomas Harrison, Stud. I.C.E.
Downey, David Gladstone, B.E. (*New Zealand*).
Ervine, Norman Derrick McKinlay, B.Sc. (*Belfast*), Grad.I.C.E.
Flemons, Kenneth John.
Fletcher, Lionel George.
Gill, Stanley, B.Sc.Tech. (*Manchester*).
Hamilton, Douglas James, B.Sc. (*South Africa*), Stud.I.C.E.
Horgan, David Noel, B.E.(*National*), Grad.I.C.E.
Jowsey, Victor, Stud.I.C.E.
Kerr, Christopher Young, B.Sc. (*Glasgow*), Grad.I.C.E.
Knox, Hugh Stuart Geddes, B.Sc. (*Belfast*), Grad.I.C.E.
Lewis, Robert Underwood, Grad. I.C.E.
Lister, Henry, M.A. (*Cantab.*).
Lougher, John Lewis, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Lovell-Smith, William John.
McCulloch, Ronald Pollock, B.Sc. (*Glasgow*), Grad.I.C.E.
Mackrell, Alexander George, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Marsh, Leonard Henry Painter, Grad.I.C.E.
Mirvis, Lionel, B.Sc. (*Witwatersrand*).
Morris, Keith Brockie Spencer, B.Sc. (*Wales*).
Noble, Edmond Denis, B.Sc. (*Belfast*).
Ralph, Gordon Bayard.
Robertson, Robert Angus, B.Sc. (*Edinburgh*), Stud.I.C.E.
Skeat, William Oswald, B.Sc.(Eng.) (*London*).
Sutherland, John Menzies, Stud. I.C.E.
Van Breda, Michiel Willem, B.Sc. (*Cape Town*), Stud.I.C.E.
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White, James Bernard, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Wilson, Henry Borthwick, B.Sc. (*Glasgow*).
Wintsch, Dennis Henry, M.A. (*Cantab.*), Grad.I.C.E.
Wood, George Alexander, B.Sc. (*Glasgow*), Stud.I.C.E.
Wrigley, Eric Albert, Grad.I.C.E.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

Charles Augustus Carlow (E. 1938).
Somers Howe Ellis (E. 1896, T. 1907).
George Fraser (E. 1913, T. 1929).
Sir John McFarlane Kennedy, O.B.E., B.A. (E. 1905, T. 1914).
Charles Edward Larard, D.Sc.(Eng.) (E. 1895, T. 1913).
Percival Arthur Rupert Leith (E. 1932).
Edward Langlois Montagnon, Ph.D, B.Sc.(Eng.) (E. 1919, T. 1938).
Henry Nimmo, C.B.E. (E. 1936).
Leslie Preston Parker, O.B.E., B.Sc.(Eng.) (E. 1918, T. 1946).
Theodore Stevens, B.Met., E.M. (E. 1904, T. 1910).
Rees John Williams (E. 1927, T. 1951).

Associate Members

Joseph George Alexander (E. 1904).
Thomas Henry Beckett (E. 1928).
William Charles Neville Bosworth (E. 1929).
Edward Mervyn Dench, B.E. (E. 1951).
John Maxwell Granger (E. 1906).
John Crabbe Lambert (E. 1951).
Robert Spence (E. 1927).
George Waters, B.Sc.(Eng.) (E. 1937).
Thomas Allen Womersley (E. 1915).

Associate

William Hanneford-Smith, F.R.S.E. (E. 1927).

Graduate

Peter Robinson (A. 1954).

OBITUARY

JOSEPH GEORGE ALEXANDER, who died at his home in Dunedin, New Zealand, on the 20th August, 1954, was born in Aberdeen on the 26th September, 1872. He received his early training in Aberdeen as an articled pupil of Messrs Walker and Duncan, and from 1895 to 1900 served as engineering assistant to the Borough Engineer's Department, Wolverhampton. For the following two years he held a similar appointment in West Ham, and from 1902 to 1913 was the Eastern Division Assistant Surveyor in the City of Westminster Engineer's Department. Early in 1913 he went to India, and after three years as Municipal Engineer in Cawnpore and six years in the Public Health Department, Bihar and Orissa, he moved still farther afield to become Municipal Engineer in Gisborne, New Zealand. In 1923 he was appointed City Engineer, Dunedin, and Engineer to the Dunedin Drainage and Sewerage Board, a post which he held from March 1924 until his retirement in March 1939.

Mr Alexander was elected an Associate Member of the Institution in 1904. He was also an Associate Member of the Institution of Mechanical Engineers, a Member of the New Zealand Branch of the Royal Sanitary Institute, and Past-President of the Institution of Engineers.

He is survived by his wife, one son and two daughters.

THOMAS BELL, E.D., who died in India on the 24th June, 1954, was born at Rock Ferry, Cheshire, on the 6th March, 1902. His technical education commenced at Birkenhead Technical College and Liverpool Central Technical School, and he was later articled to Mr B. P. Wall, who was Chief Engineer to several of the firms in the Unilever combine.

From 1924 to 1950 he served with the Bengal-Nagpur Railway, first as Assistant Engineer, later as District Engineer, Assistant Bridge Engineer, Administration Officer (Vizagapatan), and finally as Deputy General Works Manager. From 1950 until his death, he was Chief Engineer to the Bengal Baroda and Central India Railway—now the Western Railway.

Mr Bell was elected Associate Member of the Institution in 1927, and was transferred to the class of Members in 1946. He was also a Member of the Institution of Engineers (India).

During the 1939–45 war he served for three years as Major in the Indian Engineers (Defence of India), later known as Defence of India Corps (Railways). He was also Captain in the B.N.R. Battalion (A.F.I.) until its disbandment in 1947. He was awarded the Efficiency Decoration in 1953.

Mr Bell is survived by his wife, two sons, and one daughter.

CHARLES AUGUSTUS CARLOW, D.L., LL.D., J.P., F.R.S.E., who died at his home in Kincaple, St. Andrews on the 13th August, 1954, was born at Leven, Fifeshire, on the 30th November, 1878. He received his early education in engineering as a mining student at the Heriot-Watt College and Edinburgh University. He continued his studics and practical training in large collieries in Northern Durham and Lancashire, and at the

Fife Coal Company's collieries. In 1900 he was appointed Assistant Manager of the Fife Coal Company, and from then until 1911 his major responsibilities included the planning and sinking of the Mary Pit, Lochore, and the Kinglassie and Valleyfield collieries, and the establishment of all the ancillary works. From 1908 to 1914 he was responsible for the electrification of a number of collieries. In 1911 he was appointed General Manager of the Company, a post which he occupied until 1923 when he was appointed Managing Director.

In 1939 he succeeded Sir Adam Nimmo as Chairman and acted as Chairman and Managing Director until the Company went into voluntary liquidation in 1952.

Mr Carlow was elected a Member of the Institution in April 1938. He was also a former Chairman of the Scottish Transport Commission, an Honorary Doctor of Laws of St Andrews University, a Deputy Lieutenant of the County of Fife, a Justice of the Peace, an Honorary Colonel of the Royal Engineers, and an Honorary Member of the American Institute of Mining and Metallurgical Engineers. He was a Founder Member of the Institute of Fuel, Past President of the Mining Institute of Scotland, Past President of the Association of Mining Electrical Engineers, Past President of the Institution of Mining Engineers, and a Fellow of the Royal Society of Edinburgh.

DAVID HENDERSON, who died on the 26th August, 1954, as the result of an accident at Hoy Pumping Station, Caithness, was born on the 29th March, 1884.

He was educated at the High School of Glasgow, and at the Royal Technical College, Glasgow. He received his practical training with Robert McAlpine & Sons, and in 1905 was appointed Assistant Engineer to the Caledonian Railway Company.

In 1910 Mr Henderson joined the staff of the Sudan Government Railways as an Assistant Engineer. He was employed initially on a 400-kilometre railway extension, later as Resident Engineer on bridge construction, relaying, and well boring. After being promoted to District Engineer in 1918, and to New Works Engineer in 1927, he subsequently became Chief Engineer to the Sudan Railways in 1931, a position which he held until his retirement in 1934.

Mr Henderson was elected an Associate Member of the Institution in 1910, and was transferred to the class of Members in 1933.

In 1936 Mr Henderson became a Director of John McAdam & Sons Ltd., a position which he held until the time of his death. During this time his firm carried out several large public works schemes.

He is survived by a son who has now taken his father's chair in the firm.

HERBERT ROSS HOOPER, M.A., O.B.E., who died at Chippenham, Wilts, on 6th July, 1954, at the age of 90, was born at Hove on 21st April, 1864, being the 5th son of the Rev. Robert Poole Hooper, M.A.

He was educated privately and at Oxford University where he was awarded his Master of Arts degree.

In 1886 he joined the firm of Barry Brereton & Brunel as a pupil and two years later entered the engineering workshop of Messrs Westwood & Baillie in East London. From 1889 to 1891 he was engaged in harbour and railway work in Argentina, but in the latter year he went to Canada where he joined, first the Grand Trunk Railway and later the Canadian Pacific Railway, becoming Chief Bridge Inspector on the latter.

Returning to England in 1896, he entered the field of electric traction and was successively Engineer-in-Charge of the construction of the London United Tramways, the Edgware Electric Railway and the Bournemouth Electric Tramways.

In 1902, he became an Engineering Inspector of the Local Government Board, specializing at first in electrical work, and held many Inquiries into projects for the establishment of municipal generating stations. During the 1914–1918 war he was lent to the War Department in charge of the lighting, heating, and sanitary arrangements of a group of military hospitals. He returned to the then newly formed Ministry of Health after the war with the rank of Captain and the award of the O.B.E., and held the position of a Senior Engineering Inspector till 1927, when he retired after 25 years in Government service.

He then joined the late Mr George Parker Pearson in private practice in Chippenham, Wilts, and on the latter's death in 1942, he assumed as partner Mr J. B. Harvey. He retired from active business in 1952 at the age of 88.

Mr. Hooper was elected an Associate Member of the Institution in 1890, and transferred to the class of Members in 1903.

He leaves a widow, but his only son, a naval officer, died on active service during the last war.

Sir JOHN MACFARLANE KENNEDY, O.B.E., B.A., who died on the 31st August, 1954, was born on the 12th October, 1879. He was educated at Trinity College, Cambridge.

From 1908 to 1934 Sir John was a Partner in the firm of Kennedy & Donkin, electrical consultants. During the 1914–18 war he superintended the construction and operation of Government rolling mills at Southampton.

He was a Member of the Electricity Commission from 1934 to 1948, finally becoming Chairman. Sir John was also a Member of the Uganda Electricity Board from 1948 until the time of his death.

Sir John was elected an Associate Member of the Institution in 1905, and was transferred to the class of Members in 1914. He was a Past-President of the Institution of Electrical Engineers.

He is survived by his wife, Lady Kennedy, one son, and one daughter.

EDWARD LANGLOIS MONTAGNON, Ph.D., who died at his home at Chislehurst, Kent, on the 3rd September, 1954, was born in Hampstead

on the 3rd December, 1884. He graduated in engineering at London University, where he was later awarded the degree of Doctor of Philosophy for an original thesis on stresses in girders.

After obtaining practical experience in the work-shops of the South Eastern and Chatham Railway, and design and estimating experience with J. Stone & Co. Ltd, Dr Montagnon then joined Ransomes and Rapier Ltd, as an estimating designer in 1908. He was appointed Chief Engineer of the Company in 1940 and Technical Director in 1953.

Dr Montagnon's particular interest was in the design of sluice gates and water-control equipment, and he was closely associated with the design of the sluices for the heightening of the Aswan Dam.

He was the Author of a Paper presented to the Institution on "The Effect of Adding Flange-Plates to Plate-Web Girders." *

Dr Montagnon was elected an Associate Member in 1919, and was transferred to the class of Members in 1938.

He is survived by his wife and three sons.

Sir HERBERT GERAINT WILLIAMS, Bart., M.P., M.Sc., M.Eng., who died on the 25th July, 1954, at his home in London, was born on the 2nd December, 1884. He was educated at Liverpool University, gaining honours degrees in both electrical engineering and mathematics.

After an apprenticeship with Siemens Bros Dynamo Works, Stafford, Sir Herbert became assistant to a consultant marine engineer, subsequently joining a firm of public works contractors as an electrical engineer. Later he became a Director of several Companies and a consultant on economics.

Sir Herbert had been Member of Parliament for Croydon East since 1950, having previously represented South Croydon from 1932 until 1945, and Reading from 1924 until 1929. He was Parliamentary Secretary to the Board of Trade from 1928 to 1929. He was the author of several books on politics and economics.

Sir Herbert was elected an Associate Member of the Institution in 1910. He became a Knight Bachelor in the Birthday Honours of 1939, and was made a Baronet in 1953.

He is survived by his wife, Lady Williams, a son, and a daughter.

* Selected Engineering Paper No. 109, Instn Civ. Engrs, 1931.

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